PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

VOL. 62 OCTOBER, 1936 NO. 8, PART 1

TECHNICAL PAPERS

AND

DISCUSSIONS

Published monthly, except June and July, at 99–129 North Broadway, Albany, N. Y., by the American Society of Civil Engineers, Editorial and General Offices at 33 West Thirty-ninth Street, New York, N. Y. Reprints from this publication may be made on condition that the full title of Paper, name of Author, page reference, and date of publication by the Society, are given.

Entered as Second-Class Matter, December 28, 1931, at the Post Office at Albany, N. Y., under the Act of March 3, 1879. Acceptance for mailing at special rate of postage provided for in Section 1103, Act of October 3, 1917, authorized on July 5, 1918.

Subscription (if entered before January 1) $8.00 per annum. Price $1.00 per copy.

CURRENT PAPERS AND DISCUSSIONS

		Discussion closes
A Direct Method of Moment Distribution. *T. Y. Lin*	Dec., 1934	
Discussion	Mar., May, Aug., 1935	Close
Tapered Structural Members: An Analytical Treatment. *Walter H. Weiskopf and John W. Pickworth*	Oct., 1935	
Discussion	Feb., Mar., May, Aug., 1936	Close
Stable Channels in Erodible Material. *E. W. Lane*	Nov., 1935	
Discussion	Feb., Apr., May, Aug., 1936	Close
Lateral Pile-Loading Tests. *Lawrence B. Feagin*	Nov., 1935	
Discussion	Nov., 1935, Jan., Feb., Aug., Sept., 1936	Close
Sedimentation in Quiescent and Turbulent Basins. *J. J. Slade Jr.*	Dec., 1935	
Discussion	Feb., May, 1936	Oct., 193
Wind Stresses in Reinforced Concrete Arch Bridges. *A. A. Eremin*	Dec., 1935	
Discussion	Apr., Aug., Sept., 1936	Oct., 193
Successive Eliminations of Unknowns in the Slope Deflection Method. *John B. Wilbur*	Dec., 1935	
Discussion	Mar., Apr., May, Aug., 1936	Oct., 193
Reinforced Concrete Members Under Direct Tension and Bending. *D. B. Gumensky*	Dec., 1935	
Discussion	Mar., Apr., Aug., 1936	Oct., 193
Progress Report of the Committee of the Irrigation Division on the Conservation of Water	Dec., 1935	
Discussion	Apr., May, 1936	Uncertai
Tall Building Frames Studied by Means of Mechanical Models. *Francis P. Witmer and Harry H. Bonner*	Jan., 1936	
Discussion	Apr., Aug., Sept., 1936	Nov., 193
Modern Conceptions of the Mechanics of Fluid Turbulence. *Hunter Rouse*	Jan., 1936	
Discussion	Apr., May, Aug., 1936	Nov., 193
Comparison of Sluice-Gate Discharge in Model and Prototype. *Fred William Blaisdell*	Jan., 1936	
Discussion	May, 1936	Nov., 193
Behavior of Stationary Wire Ropes in Tension and Bending. *Douglas M. Stewart*	Feb., 1936	
Discussion	May, Aug., 1936	Nov., 193
Varied Flow in Open Channels of Adverse Slope. *Arthur E. Matzke*	Feb., 1936	
Discussion	May, Aug., 1936	Nov., 193
Progress Report of Committee on Flood Protection Data	Feb., 1936	
Discussion	Apr., Aug., Sept., 1936	Uncertai
Progress Report of Committee of the City Planning Division on Equitable Zoning and Assessments for City Planning Projects	Feb., 1936	
Discussion	Aug., 1936	Uncertai
Progress Report of Committee of Engineering-Economics and Finance Division on Principles to Control Governmental Expenditures for Public Works	Feb., 1936	
Discussion	Apr., Aug., Sept., 1936	Uncertai
Surface and Sub-Surface Investigations, Quabbin Dams and Aqueduct; A Symposium	Mar., 1936	
Discussion	Aug., 1936	Uncertai
Progress Report of the Committee of the Sanitary Engineering Division on Water Supply Engineering	Mar., 1936	Uncertai
Progress Report of Sub-Committee No. 2, Committee on Steel of the Structural Division on Structural Alloy and Heat-Treated Steels	Mar., 1936	Uncertai
Progress Report of Sub-Committee No. 31, Committee on Steel of the Structural Division on Wind-Bracing for Steel Buildings	Mar., 1936	
Discussion	Sept., 1936	Uncertai
Administrative Control of Underground Water: Physical and Legal Aspects. *Harold Conkling*	Apr. 1936	
Discussion	Aug., Sept., 1936	Dec., 193
Back-Water and Drop-Down Curves for Uniform Channels. *Nagaho Mononobe*	May, 1936	Dec., 193
Dynamic Distortions in Structures Subjected to Sudden Earth Shock. *Harry A. Williams*	May, 1936	
Discussion	Sept., 1936	Dec., 193
Analysis of Vierendeel Trusses. *Dana Young*	Aug., 1936	Dec., 193
Simultaneous Equations in Mechanics Solved by Iteration. *W. L. Schwalbe*	Aug., 1936	Dec., 193
Simplified Method of Determining True Bearings of a Line. *Philip L. Inch*	Sept., 1936	Jan., 193
Analysis of Continuous Frames by Balancing Angle Changes. *L. E. Grinter*	Sept., 1936	Jan., 193
The Modern Express Highway. *Charles M. Noble*	Sept., 1936	Jan., 193
Selection of Materials for Rolled-Fill Earth Dams. *Charles H. Lee*	Sept., 1936	Jan., 193
Interaction Between Rib and Superstructure in Concrete Arch Bridges. *Nathan M. Newmark*	Sept., 1936	Jan., 193

NOTE.—The closing dates herein published, are final except when names of prospective discussers are registered for special extension of time.

CONTENTS FOR OCTOBER, 1936

STRUCTURAL APPLICATION OF STEEL AND LIGHT-WEIGHT ALLOYS

A SYMPOSIUM

For Index to all Papers, the discussion of which is current in PROCEEDINGS, *see page 2*

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
PAPERS

STRUCTURAL APPLICATION OF STEEL AND LIGHT-WEIGHT ALLOYS
A SYMPOSIUM

PAGE

FOREWORD 1125
By Jonathan Jones, M. Am. Soc. C. E.

I.—MODERN STRESS THEORIES AND FATIGUE RESEARCH. 1127
Modern Stress Theories.
By A. V. Karpov, M. Am. Soc. C. E.
Tests of Engineering Structures and Their Models.
By R. L. Templin, M. Am. Soc. C. E.
Photo-Elastic Determination of Stress.
By J. H. A. Brahtz, Esq.

II.—METALLURGICAL AND MANUFACTURING ASPECTS OF FERROUS AND LIGHT-WEIGHT ALLOYS OF INTEREST IN STRUCTURAL DESIGN AND FABRICATION.. 1183
Low-Alloy Structural Steels.
By E. C. Bain, Esq., and F. T. Llewellyn, M. Am. Soc. C. E.
Stainless, High-Alloy Structural Steels.
By M. J. R. Morris, Esq.
Light-Weight Structural Alloys.
By Zay Jeffries, Esq., C. F. Nagel, Jr., Esq., and R. T. Wood, Esq.
Corrosion in Relation to Engineering Structures.
By James Aston, Esq.

Note.—Presented at the meeting of the Structural Division, Pittsburgh, Pa., October 14-15, 1936. Discussion on this Symposium will be closed in February, 1937, *Proceedings*.

PAGE

III.—STRUCTURAL APPLICATIONS OF SPECIAL STEELS.... 1251

Actual Applications of Special Structural Steels.

By V. D. Beard, M. Am. Soc. C. E.

Evolution of High-Strength Steels Used in Structural Engineering.

By Leon S. Moisseiff, M. Am. Soc. C. E.

IV.—LIGHT-WEIGHT STRUCTURAL DESIGNS................ 1303

The Application of Stainless Steel in Light-Weight Construction.

By E. J. W. Ragsdale, Esq.

Structural Application of Aluminum Alloys.

By E. C. Hartmann, Assoc. M. Am. Soc. C. E.

Magnesium Alloys and Their Structural Application.

By A. W. Winston, Esq.

FOREWORD

By Jonathan Jones[1], M. Am. Soc. C. E.

The automobile of to-day has been made possible—not merely made cheap, but actually made possible—by new types of steels, new treatments of steels, new knowledge regarding the properties of steels and how to control them, and, also, in some degree, by a corresponding development of the light structural metals. To-day's airplane also is the product of this research into, and development of, new metallic alloys. Where can be seen to-day, the airplane cloth so essential to the aviation of the World War? In the last few years, a new development has begun toward light-weight railway rolling stock based, again, on the newest metallic alloys.

In view of such rapid developments in various industries, of which the writer has mentioned three, it is incumbent upon any metal-using industry (upon none more than upon the fabricators of bridge and building frameworks and similar structures) to inquire what new economies, what new possibilities, may be anticipated in the structural field from the newer knowledge and the newer structural materials.

For the first quarter of the Twentieth Century this field was substantially stationary. Almost without exception the framed structures of that period were constructed of open-hearth steels of virtually a single simple class. The designers of a few monumental structures turned to nickel, manganese, or silicon for help, but these exceptional incidents made no difference in the average, or even in fairly impressive, bridge spans, and meant literally nothing to thousands of structural designers.

It would seem to-day as if that stationary condition is broken not to be resumed, or certainly not until after a long period of experimentation and development. New structural steels and light structural alloys are being pressed upon the structural engineer, not always, perhaps, with a very clear understanding of what he needs. Any survey of the situation, therefore, can be as of to-day only, but nevertheless it is unfortunate that neither as of yesterday nor of to-day has any such survey been undertaken, and the results arranged so that the average structural engineer may appreciate what is, and what is not, being done with new structural metals, and what may, and what may not, be "around the corner."

To fill this gap, and to perform this service, is the function of this Symposium. The specialists who have contributed papers are known to be both informed and informative; and they discuss questions such as the following:

(1) What does the newest laboratory knowledge reveal as to the important qualities of materials, of how to distinguish the best from the less suitable, of how to define, and attain in modern designs, that elusive function, the factor of safety?

[1] Chf. Engr., Fabr. Steel Constr., Bethlehem Steel Co., Bethlehem, Pa.; Chairman, Executive Committee, Structural Division, Am. Soc. C. E.

(2) What new tools for examination of stress conditions should structural engineers take seriously into account?

(3) What will variations in alloy additions, or in manufacturing practice, do to produce desirable qualities in structural steels, and at what cost?

(4) What degree of corrosion resistance is obtainable in materials subject to structural use, and can the price be paid?

(5) What alloys lighter than steel have properties that interest the structural engineer, and what seem to be the economic possibilities of their use?

(6) What has been done thus far, and what promises to-day to be done, in the actual utilization of these special structural metals?

(7) What is the objective in further development of new structural alloys, and for what can a premium be justifiably paid?

These several questions embrace one topic, that of the special structural metals of to-day and their structural use.

Note.—In order that the information imparted in the papers of this Symposium may be of the greatest possible usefulness to Civil Engineers it has been necessary to identify several of the alloys under the trade designations by which they are most commonly known. A consistent effort has been made, however, to avoid advocating special interests, with the understanding that the discussers, likewise, will confine their comments to the intended scope defined in each paper, as bearing on general classes of alloys rather than on individual proprietary types within those groups.

The term, "kips", to denote "kilo-pounds" or "thousands of pounds", has been used by each author because of the opportunity of thus arranging the tables in a more compact and convenient form; and an effort has been made to avoid conflicts in the introduction of the few algebraic symbols involved. The papers are not mathematical as a rule and only a few algebraic symbols have been required. These conform essentially to the American Standard Symbols for Mechanics, Structural Engineering and Testing Materials[2].

[2] A. S. A.—Z10*a*—1932.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

STRUCTURAL APPLICATION OF STEEL AND LIGHT-WEIGHT ALLOYS

A SYMPOSIUM

PAGE

I.—MODERN STRESS THEORIES AND FATIGUE RESEARCH:

Modern Stress Theories.
By A. V. Karpov, M. Am. Soc. C. E. 1128

Tests of Engineering Structures and Their Models.
By R. L. Templin, M. Am. Soc. C. E. 1155

Photo-Elastic Determination of Stress.
By J. H. A. Brahtz, Esq. 1171

MODERN STRESS THEORIES

By A. V. Karpov[3], M. Am. Soc. C. E.

Synopsis

Structural design is a field that has not been strictly defined. Commonly, it is assumed to include only stationary structures of a fundamental character. A broader definition would be more in line with the present scientific engineering attitude. The same basic principles that are used in the design of non-moving structures—as, for instance, bridge trusses—are applied in the design of railroad rolling stock, automobile frames, trusses of the lighter-than-air ships, or structural members of airplanes, etc. The inter-relation and reciprocal dependence of these widely different fields of engineering endeavor is growing in importance, extending structural design into fields that commonly are assigned to mechanical engineering.

Due to the experimental and theoretical investigation made during the last few decades and prompted mostly by the needs of automobile and airplane designers, engineering science has undergone a considerable change, not unlike the spectacular changes in physical and chemical sciences. These changes are influencing mechanical designs profoundly, but in so far as structural designers are concerned, there are diverging views concerning the necessity of changes in design conceptions and practice.

For quite some time the structural engineer avoided the issue by assuming that the new developments were confined to problems in mechanical engineering, and, in particular, to its most modern branches, aeronautical and automobile engineering. The rapid advance in other engineering branches necessarily must be reflected in structural engineering.

Engineering Designs as Probability Problems

Every proposed engineering structure presents a probability problem that can be stated as follows: What is the probability that the design decided upon will result in a most suitable structure during the entire period of its assumed useful life?

In its broad sense, an engineering design is an attempt to solve this probability problem by determining the future suitability of an engineering structure, considering the many governing viewpoints. In a structural design the safety of the structure is the most important consideration; economy, utility, and durability are the other viewpoints that are most often considered.

---

[3] Designing Engr., Aluminum Co. of America, Pittsburgh, Pa.

The determination of its suitability is based on the prediction of the probable future behavior of an engineering design. The difference between the predicted and actual behavior is an indication of the state of engineering knowledge at the time the design was made. This indication is of a very general nature since it encompasses not only the assumption on which the purely engineering aspects of the design were based, but also the general assumptions made, before the structure was built, concerning the future service conditions.

That every engineering structure must function during a certain period of time was always realized. The structures of ancient times in which only materials were used as they occurred in their natural state—as, for instance, natural stones—were very little influenced by aging. Such structures stood for indefinite periods of time.

At present, radically different materials are used which are much more effectively utilized and which have the tendency to change their properties either with time or with stress application or with both. As a general rule, the fact that a structure is adequate and safe under initial service conditions does not assure its future adequacy and safety. Fast-running engines in which numerous and rapid variations of stress are taking part were probably the first that called the attention of engineers, to the fact that a design that was entirely satisfactory during the initial load application may fail after a comparatively short service, due entirely to the changes in the stress-bearing capacity of the material.

The ultimate purpose of every engineering structure is to receive external and gravitational forces that must be distributed, transformed, and transmitted to some outside medium. The purpose of an engineering design is to predict these forces and the influence they will have on the structure during the entire period of its life.

Changes in Engineering Attitude as to Materials and Design Methods and Assumptions

Present and, probably still more, future engineering progress depends very largely upon the changed engineering attitude as to the materials to be used in engineering designs. Instead of simply utilizing the existing easily obtainable materials, new materials are being developed which are not so easily obtainable and, consequently, are more expensive, but which, for one or a number of reasons, are preferable, or may be even more economical, notwithstanding their higher cost.

In most applications the advantageous and economical use of such materials requires a better utilization than is customary with less expensive materials. The improvement in utilization is attained by a better understanding of the properties of the materials and closer evaluation of the future behavior of the structure.

In the past, for instance, it was considered good engineering to build machinery that was able to carry a much higher load than was specified, reflecting the insufficient engineering knowledge with respect to the requirements

and actual performance. With reference to bridges, even at present, the claim is often made that a particular bridge is well designed because it carries loads that are much heavier than were originally specified.

The structural designer realizes only gradually the up-to-date viewpoint of the aeronautical, mechanical, or electrical engineer, that good engineering requires a design that fits the conditions. That an over-ample design is poor engineering as well as an inadequate one. In that connection one of the interesting recent changes in engineering ideas is the attitude toward weight. A few years ago the weight was considered not only as a favorable factor in engineering designs, but as a criterion of quality—the heavier the better. Consequently, no particular efforts were made to reduce the weight. At present, the attempts can be followed through all fields of engineering development to create more economical or better suitable design by the elimination of useless weight: The use, if possible, of arch dams instead of gravity dams; the substitution of artificial materials in housing, which are either lighter or which can be used in less quantities; the substitution of high strength or light-weight alloys in metal structures; the introduction of welding; the lightening of bridge floors; the reduction of weight of automobiles and airplanes; and the use of weightless agencies in the numerous applications of electric energy.

The Extent of Present Theoretical Knowledge

The present electronic theory is based on the assumption that the electric and magnetic forces of the electrons are ultimately responsible for the continuity of each physical body. The different orientation of these forces is responsible for the properties of the different materials and in particular for their stress-carrying capacities. The fundamental approach to the stress problems would be to consider these electric and magnetic forces. At the present inadequate state of knowledge that approach is impractical. Even the less fundamental relations between the individual particles or crystals of the material have not been sufficiently studied and rationalized to be practically applicable.

The present engineering approach necessarily must be simplified, which is done by the introduction of the theory of stress. This theory should make it possible to determine the distribution of forces applied to the structure and their transformation into strains and stresses. It should provide a method by which the behavior of the structure may be predicted if the applied external forces or loading are known or can be assumed with sufficient accuracy. In the engineering sense of the word, stresses are changes in the internal forces that are holding these particles together. Stresses introduced in any physical body are manifested in strains and resulting deformations and deflections. The mathematical expression of the engineering theory of stress is given in a set of differential equations. The mathematical solution of these equations should result in a set of functions that can be solved for each point of the body, giving, for each set of conditions, the corresponding position of each point and the stress in each desired direction. As a general rule, the

mathematical functions are applicable only within the range of continuity and cannot be extended beyond a discontinuity.

The surface of each structural element will be a discontinuity, regardless of whether it is exposed to the atmosphere or to a liquid, or is a jointing or connection area of two structural elements or the border area between materials differing in their properties. In the engineering sense that would mean that each set of functions will determine the conditions within the body of a continuous structural element, and also at its surfaces or discontinuity areas, the latter being of particular importance. The conditions at such surfaces are referred to as "boundary conditions".

Present mathematical knowledge is limited and no general solution is available of the differential equations set up by introducing the theory of stress. Consequently, the solution of each structural problem resolves itself into finding the partial or approximate solutions of the general equations which will solve, with more or less accuracy, the particular stress problem.

All the approximate methods are based on a number of more or less reliable assumptions. Besides the traditional assumption that the material follows Hooke's law nearly all methods are based on the assumption of linear stress distribution throughout the thickness of the body. In many cases, such an assumption may be reasonable at a certain distance from the boundaries, but as a general rule it violates, grossly, the actual conditions at close proximity to the boundaries. As a consequence these methods, in general, do not satisfy the boundary conditions, although they may permit a reasonably close determination of conditions within the thickness of a uniform body.

At the boundaries, however, the difference between the theoretically determined, and the actual, conditions may be appreciable, particularly at places where a sharp change in the boundary conditions occurs. Such change may be due either to an abrupt change of cross-section or to local application of concentrated forces.

The theory of stress gives two major criteria for the determination of the suitability of a design—the deflection and the stress. The deflection is an integration of the differential strains over the entire structure or over an entire structural element. Consequently, if the average conditions determined by the use of an approximate method are close to the actual average conditions, there may be a very satisfactory agreement between the deflections determined by the use of such a method and the actual deflections, notwithstanding the discrepancies at the boundaries.

In so far as the stress is concerned, the suitability of a structure should not be judged by the average but by the maximum stress that may occur at any point, and that maximum, as a general rule, will occur at the boundary. The value of such governing maximum stress cannot be determined with reasonable accuracy by the use of the traditional approximate methods.

Properties of Metals and Alloys of Importance in Structural Designs

The ideal material, which is the basis of conventional designs, not only has perfect elastic properties, obeying Hooke's law through the entire range of stress, and exhibiting a linear stress distribution through the thickness of

the body, but it is also unaffected by time, retaining the same properties after an indefinite length of service. No actual material has such properties and in up-to-date structural design the behavior of the material must be considered during the first, as well as the intermediate and last, load application for the entire expected life of the structure. The possible deterioration of the material with time, the creep, and the fatigue properties, must be investigated before an engineering judgment can be formed as to the suitability of the particular material for a particular application.

The stress-resisting capacity of a metal or alloy is the fundamental property that makes possible its use in any engineering structure. The suitability of different metals and alloys is judged by comparing their ultimate strength, elastic limit and elongation, fatigue, creep, impact properties, weight, and price.

Metal and Alloys Used.—The pure metals are very unsatisfactory in so far as their stress-resisting capacity is concerned. No pure metals are used in structural designs; the carbon steels are the oldest and most important alloys. The modern steel alloys, the aluminum and magnesium alloys, are recent additions to the structural field.

From the seructural designer's viewpoint an ideal alloy should possess high stress-carrying capacity at light weight and low cost and, at the same time, should have a high degree of permanence or a high degree of corrosion resistance. A series of alloys having these qualities and of different moduli of elasticity would make it possible to use the most suitable alloy for each design.

No alloy or alloys are available at present that would meet all the requirements satisfactorily. The ultimate goal of metallurgical research is to find compositions and fabrication methods that will improve the stress-resisting as well as the other desirable properties. Such an ideal alloy is not even in sight, and structural engineers will be compelled, for a long time to come, to use numerous alloys, each one satisfying a part of the requirements among which the cost will always be important.

In so far as the modulus of elasticity is concerned, it seems that, at present, metallurgical science does not propose any methods by which that important property can be varied sufficiently.

Ultimate Strength.—Ultimate strength is measured by the stress at which the material fails during a single gradually increasing load application, based on the original cross-section of the specimen and using an arbitrary size and shape of specimen. One of the complications in the application of present stress theories, lies in the fact that the ultimate strength depends not only on the kind of material, but also on the kind of stress. The same material will show a different ultimate strength in tension, compression, bending, or shear.

Considering the simplest case, the theoretical maximum tensile strength of a material reflects the degree of adhesion between its molecules.

The general assumption is that this adhesion is due to the molecular or internal forces that hold the molecules of the material together. If the modern

electronic theories are accepted, it is only logical to assume that there is a particular arrangement of all electrical and magnetic forces that will result in the maximum internal force holding the particles together and, consequently, will represent the conditions of the absolute ultimate tensile strength that cannot be exceeded by any material under any conditions. Then the ultimate tensile strength obtained in any material under a particular set of conditions could be expressed as a percentage of the absolute tensile strength. This percentage will be low for unsatisfactory arrangements of the internal forces and will increase if the internal forces are better orientated with reference to the tensile stress applied. No information is available as to what such absolute ultimate strength may be, but it is reasonable to assume that it is not even approached in any of the commercially used structural materials.

The present state of knowledge is insufficient to determine how a better orientation of the internal forces may be obtained, but practical experience shows that such improvements are possible even if an explanation for them is not apparent. It is not even clear whether the increase in the specific gravity of the material has any favorable influence on the orientation of the internal forces.

The most important improvement in the orientation of the internal forces, resulting in the increase of ultimate strength, is obtained by cold working suitable alloys. At the same time such cold working not only does not increase, but very often decreases, the density of the material. This fact could be taken as an important indication that the proper orientation of internal forces has no direct relation to the specific gravity, but depends on some factors unknown.

The general speculation permissible under these assumptions is that it should be possible by the use of means, unknown at present, to produce alloys of the same degree of orientation of internal forces irrespective of their specific gravities. Practically, that would mean that it should be possible to produce light alloys of very high ultimate tensile strength.

Materials identical in so far as the orientation of the electric and magnetic forces is concerned, should be identical in all other respects. There are numerous ways in which the orientation of these forces can be changed in metals, resulting in an unlimited variety of metals and alloys; but practically nothing is known as to the character of these re-orientations. The following classification of the known methods of re-orientation of the internal forces may be made:

I.—Re-orientation involving change in chemical composition:

- (*A*) Changes brought about by the addition or substitution of different alloying elements, resulting in alloys of a different chemical composition.
- (*B*) Changes in composition of the integral parts of the alloys, resulting in alloys of the same general chemical composition, but of different properties.
- (*C*) Changes in surface conditions, known as chemical corrosion.

II.—Re-orientation not involving any change in chemical composition:

(*A*) Changes in grain structure caused by cold working or heat treatment.
(*B*) Changes in the internal equilibrium conditions caused by the introduction of internal stresses.
(*C*) Changes in surface conditions caused by mechanical erosion.
(*D*) Changes in surface conditions caused by repeated application of stress.

Factors that do not influence the stress-resisting capacity directly, but are customarily included, may be classified as follows:

III.—Factors reflecting insufficient theoretical knowledge:

(*A*) Stress concentrations that cannot be determined exactly.
(*B*) Unknown distribution of stress throughout the thickness of the specimen resulting in the introduction of a new variable—the size of the specimen.

IV.—Factors reflecting inadequate manufacturing procedure:

(*A*) Non-uniformity of material.
(*B*) Unsatisfactory surface conditions.

The chemical composition is naturally the major factor. Nothing is known concerning the fundamental relation between chemical composition and strength of the various materials and in particular metals. Much has been learned during the last few decades about the influence of comparatively small quantities of some alloying elements on the strength of commercial alloys. Nevertheless, the development of new alloys is very much a "cut and try" proposition. The present metallurgical knowledge of the fundamental factors influencing the strength of metals is rather limited. In general, every commercially used metal can be made stronger by adding proper alloying ingredients. Even the impurities that are unavoidable in any commercial process may change the strength of metals to a large extent.

On the other hand, in most cases, the increase of strength indicated by increased ultimate strength that is due to alloying, is obtained with a simultaneous decrease in ductility, indicated by decreased elongation. Since, for practical purposes, a metal of high strength and high ductility is desirable, it is necessary to arrive at some compromise as to the most favorable combination of strength and ductility. For alloys to be used as castings it is possible to utilize materials of very low ductility; metals that must be changed to a prescribed shape by forging , stamping, extrusion, or rolling necessarily must have a much higher ductility, at least at the working temperature.

Some of the alloys are susceptible to heat treatment and change their properties if heated to a certain temperature and cooled in a definite way. The wide influence that changes in chemical composition may have on alloys with different percentages of alloying elements is demonstrated in Fig. 1, which shows the total percentage of alloying elements and the ultimate tensile strength of a number of non-commercial and commercial aluminum alloys. It embraces the highest values of the ultimate tensile strength obtained.

In drawing this diagram, no attention was paid to the method by which the highest ultimate strength was obtained, such as heat treatment, cold working, etc. The greatest tensile strength obtained for each alloy was the

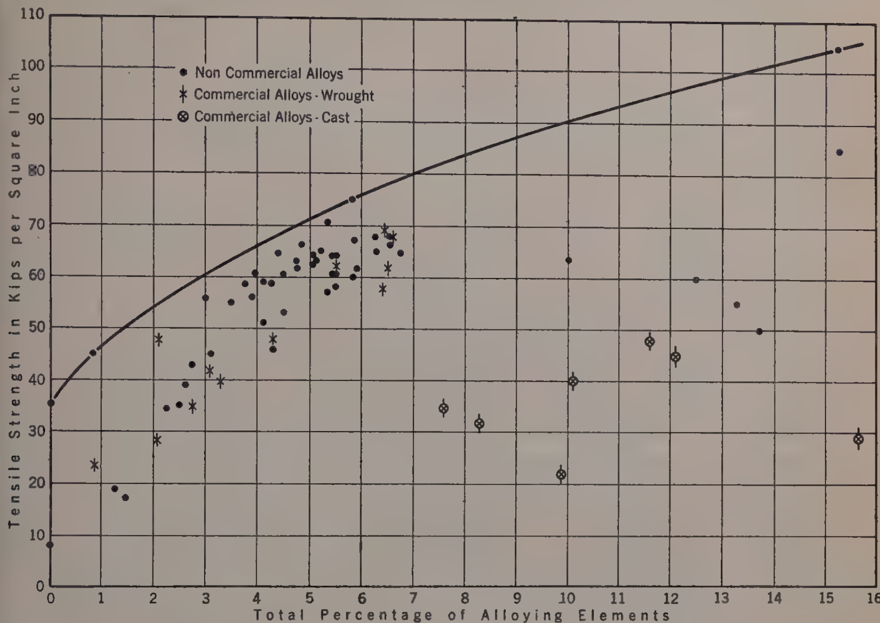


FIG. 1.—ULTIMATE TENSILE STRENGTH OF ALUMINUM ALLOYS.

information included in Fig. 1. It is reasonable to assume that every point of the enveloping curve can be filled in, with future progress in the study of the different possible alloys.

The increase in the percentage of alloying elements increases the ultimate tensile strength, but may decrease the ductility, and in many cases the corrosion resistance, to such an extent that the resulting alloys often have no commercial use. Increasing the number of alloying elements may improve these conditions, resulting in increased stress-resisting capacity without a detrimental influence on other desired properties, resulting in commercial alloys.

The large number of points representing non-commercial alloys indicates that a large number of factors needs to be considered in order to find whether a particular alloy will prove to be useful.

No information is available that would indicate the maximum strength that may be reached by the alloying of different base metals.

Stress-Strain Curves, Modulus of Elasticity, Yield Point.—Although very little is known about the ways in which the different chemical ingredients act in the alloy, the final results are reflected in stress-strain curves, particularly if these curves are obtained by the use of sufficiently sensitive instruments.

A number of such curves are shown in Fig. 2, which is drawn in such way that for the first part of the stress-strain curves, up to 0.01 in. per in., or 1% of strain, a large scale of strains is used, the remainder of the diagram is

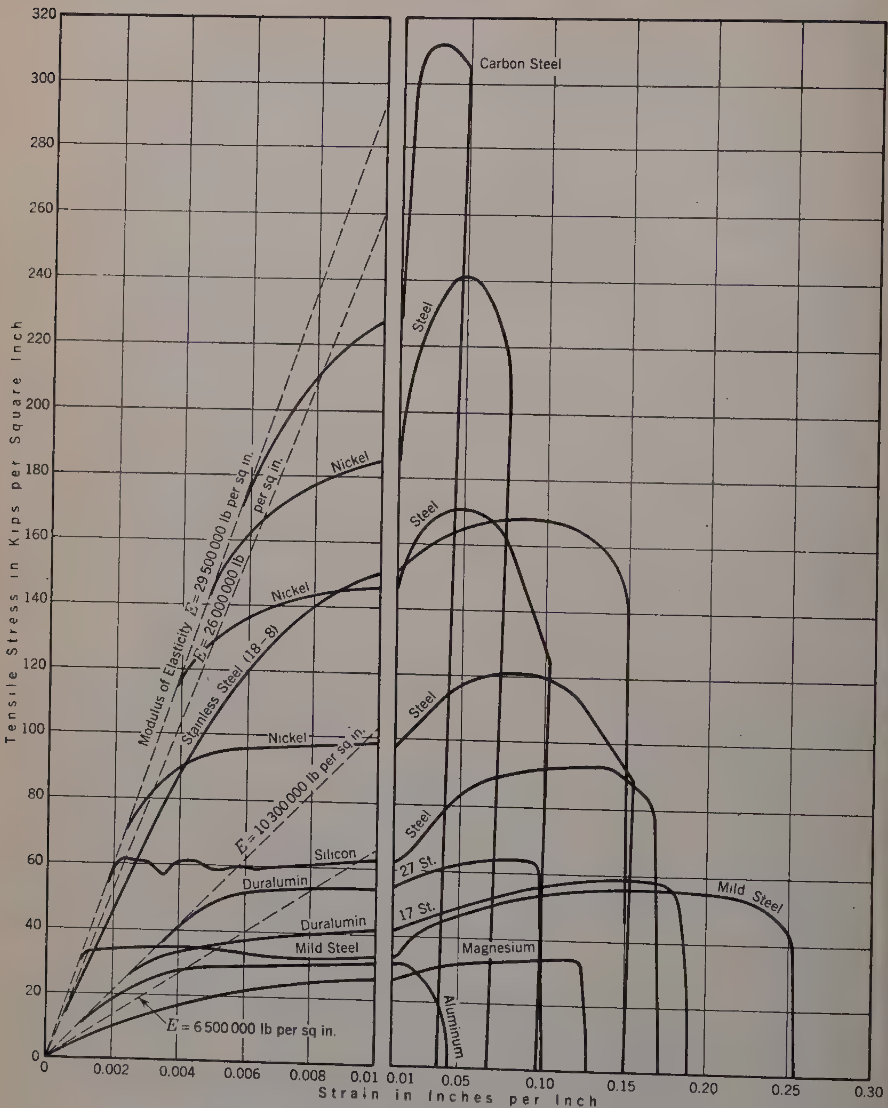


FIG. 2.—COMPARATIVE STRESS-STRAIN CURVES

drawn, using a strain scale one-twentieth of the size of the original one. This diagram, based on publications of V. D. I. (Verein Deutscher Ingenieur), and unpublished tests made by R. L. Templin, M. Am. Soc. C. E., shows the three types of commercial alloys that are used or may be used in struc-

tural designs, the steel, aluminum, and magnesium alloys. A study of Fig. 2 reveals a few interesting facts:

1.—Notwithstanding the very large variation of the ultimate strength and elongation, only four different moduli of elasticity are available, one between 29 000 and 30 000 kips per sq in. for low alloy steels, the other between 25 000 and 26 000 kips per sq in. for high alloy, stainless, steels, the third about 10 300 kips per sq in. for aluminum alloys, and the fourth about 6 500 kips per sq in. for magnesium alloys. In structural materials as commercially used at present, no intermediate moduli of elasticity are available.

2.—Yield points may be studied in Fig. 2. The low-strength steel alloys—the mild steel and the silicon steel—have pronounced yield points at which a definite break of the strain-stress curve occurs. The high-strength steel alloys as well as the aluminum and magnesium alloys have no definite yield points, the stress-strain curve changing from the straight part to a curved part gradually and steadily without any break.

The importance of the yield point for mild steel and particularly for silicon steel is obvious. Any stressing of these alloys above the yield point will result in considerable deformation that must take place before the increase of resistance will occur that may stop such deformation.

For alloys which do not have yield points it is customary nevertheless to designate as yield point the stress at which an arbitrarily chosen permanent deformation occurs. For non-ferrous alloys, 0.2% of permanent deformation is widely used. For ferrous alloys a permanent deformation between 0.1% to 0.5% is usually chosen.

Considering the properties of the alloy and its applications, these arbitrary yield-point values are rather meaningless, but they have some practical significance in limiting the acceptable design deformations.

3.—Fig. 2 makes clear the justification of the application of Hooke's law to low-strength steels if they are not stressed above the yield point; also, it demonstrates the much lesser justification for the application of this law to high-strength steels and light-weight alloys and the non-applicability of Hooke's law to low-strength steels stressed above the yield point.

4.—It shows: (*a*) The uncertain nature of the definition of the modulus of elasticity; and (*b*) the difference between the initial and the actual moduli of elasticity for the different types of alloys.

Boundary Conditions.—The surfaces of any structural element are boundaries which represent the most abrupt change in the conditions. In structural engineering, as well as in many other engineering fields, the boundary conditions may become of considerable importance. They must be considered in any investigation of the stress-carrying capacity of alloys and should be reflected in the mathematical treatment of the stress problems. The present progress in engineering design, as well as in the utilization of alloys, depends to a large extent on the increased attention paid to the boundary problems.

That corrosion, or the disintegration, of metals and alloys starts at the boundary or surface and gradually penetrates the body of the material was

realized, of course, a long time ago, and the metallic surfaces were protected by the use of different paints, or, more recently, by facing the less stable base metal with a more stable alloy or alloying element.

Even now the influence of the boundaries on the distribution of stress within the body of the metal is not always realized. For instance, the traditionally assumed linear distribution of stress does not represent the conditions close to the boundaries. During the last few decades considerable advance has been made in realizing the importance of the boundary or surface conditions in the fatigue phenomena. Without attempting to go into the complicated stress relation among the individual crystals or even more

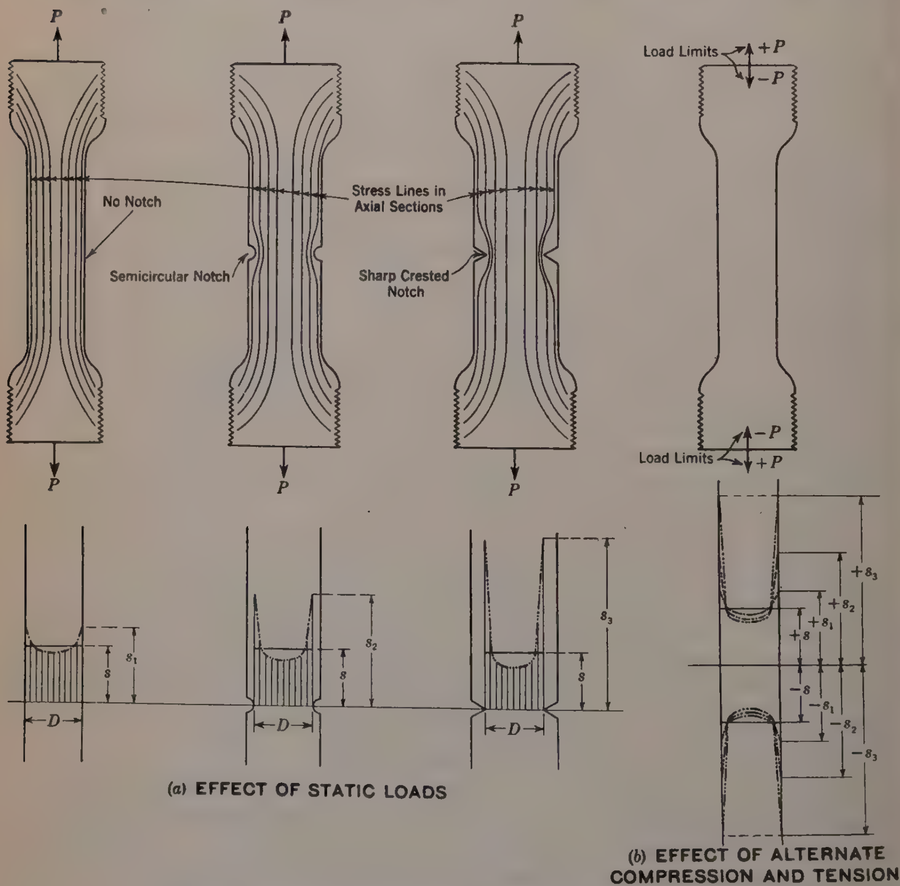


FIG. 3.—STRESS DISTRIBUTION IN CYLINDRICAL SPECIMENS.

minute particles of the material, but keeping within the customary engineering conception of stress, a simple illustration may be of interest. The stress conditions of a circular bar subjected to pure tension or compression are shown in Fig. 3(*a*) and Fig. 3(*b*) for static and varying loads, respectively.

The two plain bars have the same diameter, D, and the notched bars have the same diameter, D, as the plain bars, at the bottom of the notch. The static load, P, to which the bars in Fig. 3(a) are subjected, is the same. The varying load, P, to which the bar in Fig. 3(b) is subjected, alternates between $\pm P$, the numerical value of P being the same as in the case of the bars subjected to static loads.

Under the traditional assumption of linear distribution of stress, a tensile stress, s, will be developed at the middle section of the plain bar (Fig. 3(a)) and at the section taken at the bottom of the notches at the bars. This stress will be equal for each of these three bars and will be distributed uniformly over the entire available cross-section, as shown by solid lines in Fig. 3.

The actual stress in the plain bar (Fig. 3(a)) under static load probably will be greater at the surface and less at the middle as shown by the broken-line curve, the ratio between the maximum actual stress, s_1, and the traditionally assumed stress, s, being usually referred to as the stress concentration factor. The actual stress in the bar with the circular notch will be greater at the bottom of the notch and somewhat less at the middle, as compared with the plain bar, and as shown by the broken-line curve, the maximum stress being s_2. Finally, the actual stress in the bar with the sharp notch will be still greater at the bottom of the notch and somewhat less at the middle, and as shown by the broken-line curve, the maximum stress being s_3.

A much better visualization of the conditions may be obtained by the use of the stress fields or stress lines analogous to the fields and lines used in the study of electrical phenomena. Instead of visualizing the stress at one cross-section only, it is possible to visualize the flow of stress. In Fig. 3(a) such stress lines are drawn for an axial section of each cylinder. At each point the direction of the stress is a tangent to the stress line. The intensity of the stress is shown by the distance between the stress lines. The influence of the differently shaped notches may be visualized by the study of these diagrams.

Introducing the stress concentration factors, c_1, c_2, and c_3, the maximum stresses, s_1, s_2, and s_3, may be expressed as $s_1 = c_1 s$; $s_2 = c_2 s$; and $s_3 = c_3 s$. The values of c_1, c_2, and c_3, and, consequently, the values of the maximum stresses, s_1, s_2, and s_3, depend on the properties of the material, particularly on the shape of its strain-stress curve. As long as the material follows Hooke's law at the low range of stress, the strain-stress relation is expressed by the straight part of the curve. Under these conditions the concentration factors reach their highest value. When the material deviates from the straight part of the curve, increased deformation or plastic flow takes place. This tends to redistribute the material, tending to decrease the peak value of the stress and, consequently, to decrease the concentration factor.

Alloys having straight-line stress-strain curves and definite yield points will develop the maximum stress concentrations with increase of load until the maximum stretch reaches the yield point, after which the stress concentration factor will decrease rapidly.

Alloys that gradually diverge from the straight part of the stress-strain curve will develop lower stress concentration factors at the lower range of stress.

Assuming, again, linear stress distribution, the traditionally evaluated stress in Fig. 3(b), under the varying load, will fluctuate between plus and minus s; the numerical value of s again being the same as in the bars subjected to static load, and being uniform over the entire cross-section. The actual stress in the bar will depend on a number of conditions. If the force, P, and, consequently, the actual maximum stress, s_1, is small, the stress in the bar will fluctuate between the limits shown by broken-line curves, corresponding to the actual stress of the bar in Fig. 3(a), without producing any changes in the material. Consequently, the reversing load may be applied an indefinite number of times, without producing a failure. If the stress, s_1, is great enough, the actual stress of the plain circular bar (Fig. 3(b)) will fluctuate between limits that will change gradually. At the first load application, the stress limits will be the same as before. If the reversing load is applied a number of times, incipient cracks will be formed at the surface of the bar, an increase of the surface stress will follow, and the stress limits will be approximately the limits of the bar with the circular notch (Fig. 3(a)), shown by the broken-line curve, with the maximum stress, s_2.

In this case, again, the phenomenon may stop if s_2 is low enough and the reversing load may be applied an indefinite number of times without producing a failure; but if s_2 is great enough the repeated application of the reversing load will cause the incipient cracks to change their shapes, increasing the surface stress until it corresponds to the conditions of the bar with the sharp notch in Fig 3(a), with the maximum stress, s_3, the notch being chosen so as to produce the highest possible value of c_3.

These conditions will be somewhat complicated due to the fact that the circular bar will contract or expand, not only parallel to the axis or in the direction in which the forces are applied, but also along the diameter or at right angles to the direction of the forces. This complication, however, in the simple case assumed, will only be of secondary importance.

If the stress, s_3, developed under such conditions is low enough, no more changes in the conditions of the incipient cracks will occur, and the specimen may be subjected to an indefinite number of load reversals without failure.

If the stress, s_3, is high enough, a permanent and continuous change in the crack conditions will be produced and the failure will occur after a number of load reversals during which the incipient crack will grow and attain larger dimensions, causing an increase of stress due to the reduction of the effective cross-section. It may take a considerable number of load reversals before the incipient cracks will come to such a condition that they start to grow; but after that the final growth is usually accomplished during a small number of load applications, and the failure takes place rapidly.

Finally, the reversing load, P, applied to the specimen may be so great that although the stress, s_1, will be lower than the ultimate strength, the stress, s_3, exceeds it. Under such conditions the failure will occur after a limited number of load applications as soon as the growing maximum stress approaches the ultimate strength of the material.

This description of the fatigue phenomena may not be exact in every case, but should give an understanding of the underlying conditions as known at present.

The properties of metals and alloys of importance in engineering structures may be divided into two classes: The first are the properties that depend mainly on the internal deep-seated conditions; and the second those that depend upon the surface conditions. Modulus of elasticity, shape of the stress-strain curve, ultimate strength under static load, creep, and single impact properties, all depend on the internal conditions and are affected very little by the surface conditions. Corrosion resistance, fatigue strength, and fatigue strength under repeated impact, are properties that depend fundamentally on the internal conditions, but may be influenced to a very large degree by the surface or boundary conditions.

Fatigue Properties.—Fatigue strength is determined by the formation of incipient cracks on the surface and by the extension of such cracks. If the formation and extension of these cracks can be retarded, the fatigue strength will be increased, and *vice versa.* The stresses developed at the surfaces are the major factors in the formation of the cracks. It is not the mean stress or the assumed stress that is of importance, but the actual maximum stress that may occur.

The extension of cracks depends on the load or stress fluctuation. A static load may develop minute cracks, but unless the maximum stress is close to the ultimate strength of the material such cracks will not grow. The variation in stress will change the conditions of the cracks. Small variations of stress from a constant mean value will make the cracks spread only if the mean value is high. Larger stress variations will make the cracks spread even at a smaller mean value until the complete reversal of stress may result in their extension and spread, although the mean value is zero.

These conditions may be summarized on a diagram with a base line of 45°, such as that shown in Fig. 4, in which the mean value of the stress is plotted on the 45° base line, using either the scale on the ordinate or the abscissa axes. The stress variations are plotted vertically as ordinates using the mean stress value as the zero point. This diagram, based on the publications of V. D. I., extends from the zero mean value which will be the origin of the co-ordinate system and to the value of the ultimate strength plotted at the 45° base line.

To draw such a diagram the fatigue strengths must be determined on a large number of identical specimens of a particular alloy under different conditions of load variations, but for a stress of the same kind, either tension or compression.

For practical purposes a diagram such as Fig. 4 should not be used above the true or assumed yield-point value, resulting in that part of the diagram shown by solid lines.

Number of Load Applications.—The fatigue strength of a material is determined by the testing of a sufficiently large number of identical specimens. Each specimen is tested in a similar manner but at a different stress range.

If, for instance, the fatigue strength at the complete reversal of stress is to be determined, the specimens are tested under conditions of complete stress reversal, starting with a number of specimens that are tested at high stress.

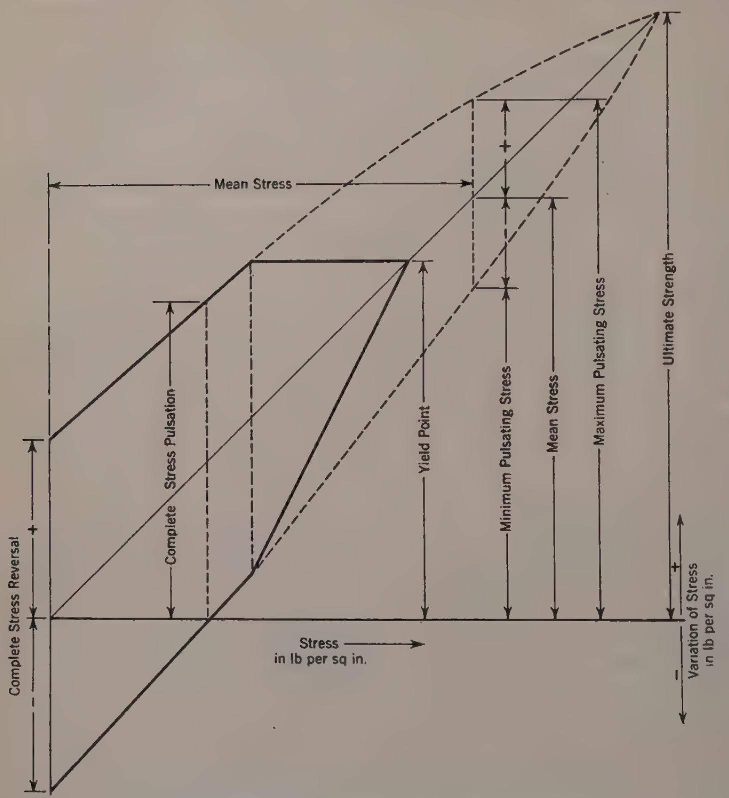


FIG. 4.—FATIGUE DIAGRAM; 45 DEGREE BASE LINE.

The testing is continued, each succeeding specimen or set of specimens being tested at a lower stress, so that the result represents a series of gradually decreasing values of stress to which the specimens were subjected. For each specimen two values are obtained, the stress at which the specimen was tested and the number of stress reversals at which it failed. The stress is determined by the use of traditional methods, disregarding any possible stress concentration.

In order to obtain complete fatigue data, tests must be made and diagrams plotted for complete and partial stress reversals and for complete and partial stress pulsations.

The accepted term, "endurance limit", can be defined as the fatigue stress under complete stress reversal below which metal will withstand, without failure, an indefinitely large number of cycles of stress. It may also be defined as the stress value at which the stress-cycle curve changes to a line, straight and parallel to the abscissas. For partial stress reversals and for pulsating stress the term, "fatigue strength", would seem more applicable.

There is a wide difference in the shape of the stress-cycle curve for different materials. For some materials the curve has a very pronounced "knee" and the endurance limit is a very definite term, but for other materials the curves have no definite "knee" and, for some materials, the stress-cycle curve does not change to a line straight and parallel to the abscissas.

To determine the properties of the different alloys it is necessary to run the fatigue tests either until a definite endurance limit or fatigue strength is established or until the proof is obtained that no definite value can be obtained.

The term "endurance limit", is often misused. For practical purposes, particularly in structural problems, data may be utilized which represent results of tests obtained at a lower number of load applications; such data should not be referred to as "endurance limit", but as "fatigue strength", and it would seem necessary in each case to supply a clear indication of the limiting number of load applications at which the data were obtained. The term, "endurance limit", should not be applied to cases of partial stress reversals or stress pulsations.

Surface Conditions.—Fatigue strength may be influenced to a large extent by changes in the surface conditions. The methods by which such changes may be obtained can be divided into a number of groups. The most radical is the group involving the change in the chemical composition of the boundary layer of the metal or alloy. If such a change results in a surface which prevents the development of cracks, or which prevents the minute cracks from spreading, an increase in fatigue strength may be obtained. The nitriding of steel changes the chemical composition of the surface, resulting in a considerable increase in the fatigue strength.

Changing the surface conditions without changing the chemical composition covers a group of methods of primary importance. First in this group is the case in which changes occur automatically during the repeated application of the fluctuating load. The changes produce what is known as the "strain-hardening effect." It seems that if, during the load application, stress concentrations occur, that bring the peak of the stress above the yield point for materials that have a yield point, plastic deformations are caused, thus changing the properties of the material that may retard or even stop the growth of the incipient cracks. For materials that do not have a yield point, the same condition occurs if the stress is high enough to produce a sufficient deformation.

Similar results may be produced by cold working the surface by use of rollers or by similar means. The increase in the fatigue strength in this case may be attributed partly to the compacting of the surface particles and partly to the pre-stressing of the surface which, if properly applied, may reduce the stress peaks.

The next group of methods involves the finish of the surfaces. Machined and highly polished specimens have the highest fatigue strength in this group. The elimination or reduction in size of the incipient minute cracks is the probable explanation. Fatigue strength is reduced if the surfaces are only

machined but not polished, and it is still more reduced for rough, non-machined surfaces. The notching of the surface, or injury by sharp cuts, and, finally (for mill-rolled shapes), using the material with the mill scale left on, will result in the lowest fatigue strength. The presence of cracks and high stress concentrations are the most probable explanations.

The final and lowest in fatigue strength is the group in which the surfaces are covered with liquids. It would seem that if liquids of high viscosity are pressed into the incipient cracks, or liquids of low viscosity flow freely into them, they act as wedges during the closing of the crack, increasing considerably the stress concentrations and reducing the fatigue strength. The chemical reactions of some of the liquids cause a further lowering of the fatigue strength by increasing the sizes of the incipient cracks or deepening them.

Finally, a very considerable lowering of the fatigue strength and the destruction of the metal, submerged for instance in water, may occur due to cavitation and resulting corrosion, when the material is subjected to a very large number of very rapid blows due to the continuous and rapid forming and collapse of vacuum bubbles in the surface layer of the water adjoining the metal.

If similar metals or alloys are compared, those that have a lower ultimate strength can be improved as a general rule to a larger degree; and, on the contrary, high-strength alloys are more sensitive in so far as the lowering of the fatigue strength is concerned. Thus, the fatigue strength of a low-strength carbon steel can be improved by nitriding to a larger degree than the fatigue strength of high-strength steel; but if compared with machined and

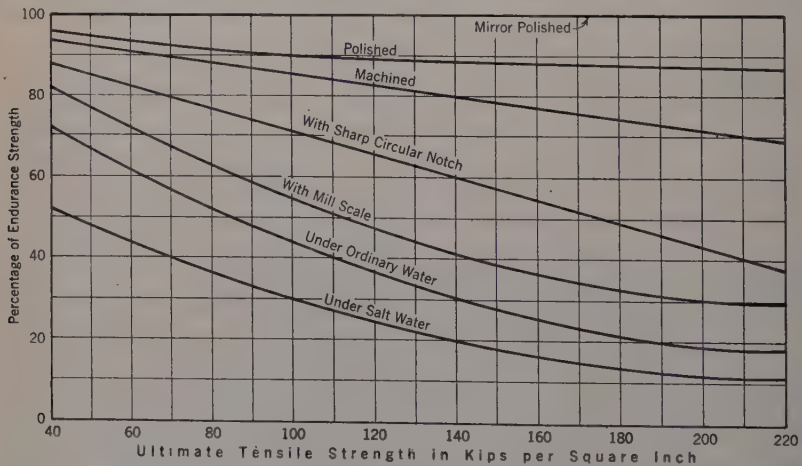


FIG. 5.—REDUCTION OF ENDURANCE STRENGTH OF STEEL ALLOYS DUE TO SURFACE CONDITIONS.

polished surfaces, a notch or surface injury will lower the fatigue strength of high-strength alloys to a larger degree as compared with similar low-strength alloys.

In so far as steel alloys are concerned, Fig. 5, based on V. D. I. publications, illustrates this conditions. The diagram is based on fatigue strength obtained

on specimens in tension-compression and bending. The highly polished or mirror-polished specimens are given the 100% rating.

Reduction factors are to be applied, depending on the ultimate strength of the particular steel alloy and its surface conditions. They must be applied not to the mean value of stress but to the plus-minus stress fluctuation. If a combination of factors is involved the lowest values shown on Fig. 5 should be used. As usual, the values for sharp circular notches do not take into consideration the stress concentrations and give a good idea as to what the reduction in fatigue strength may be if the surface is injured by a blow leaving a sharp indentation.

The influence which that reduction of fatigue strength may have under conditions of complete stress reversal is demonstrated in Fig. 6, based on the data

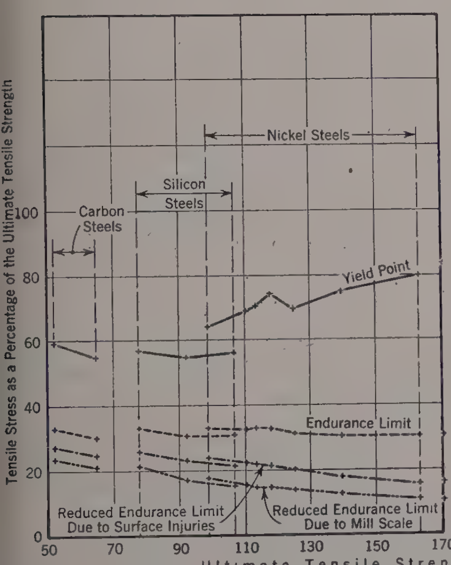


Fig. 6.

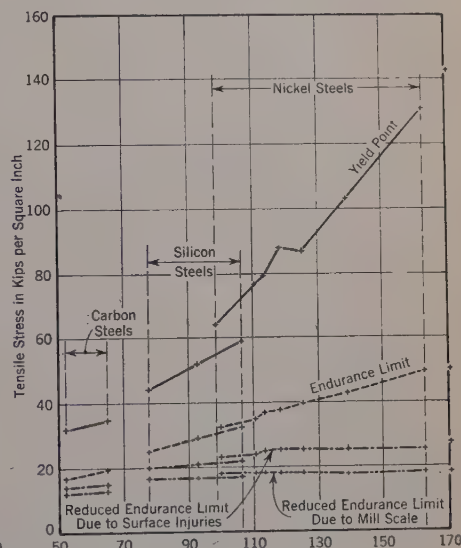


Fig. 7.

shown in Fig. 5, for different steel alloys. This diagram shows the yield point, the endurance limit for highly polished specimens, the reduced endurance limit due to surface injuries using the data for sharp circular notches, and the reduced endurance limit that is obtained if rolled material is tested with the mill scale left on. All these data are expressed in percentages of the ultimate tensile strength. Finally, the diagram, Fig. 7, shows that under the same conditions of a complete load reversal the high-grade steels have endurance limits somewhat proportional to their higher ultimate strength, only for highly polished specimens. For specimens with injured surfaces or with the mill scale left on, there is only a very slight increase of the endurance limit.

Complete Fatigue Diagrams.—The fatigue strength of a material, tested under identical conditions in so far as the properties of the alloy and its surface conditions are concerned, depends on the kind of applied stress.

Therefore, it becomes necessary to test the materials separately for three kinds of stresses—tension-compression, bending, and shear. Fig. 8, based on V. D. I. publications, shows a complete fatigue diagram of a silicon steel alloy on which the fatigue strength for all three kinds of stresses are shown. The characteristics of this alloy were: Carbon, 0.25%; silicon, 0.2%; manganese, 0.6%; ultimate strength in tension, 64 kips per sq in.; and, elongation, 0.20% in 4 in. In Fig. 8 the tension-compression data are presented for the case only where the mean stress is zero or tension. If it is desirable to include data for tension-compression when the mean stress is compression, the diagram may be extended by continuing the 45° base line into the compression region in the third quadrant.

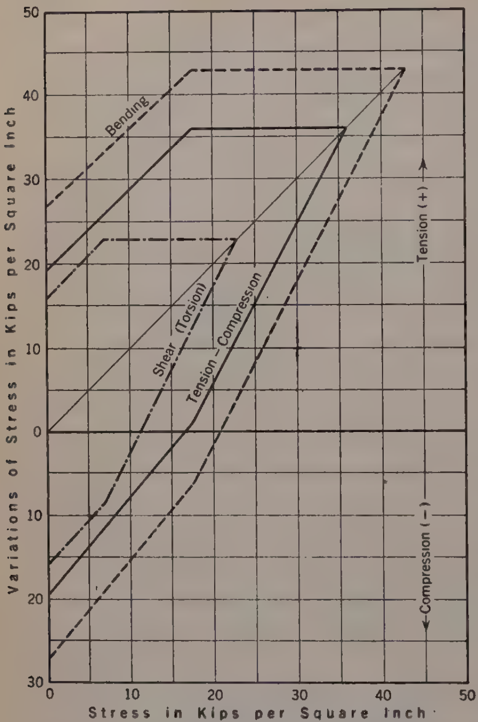


FIG. 8.—FATIGUE DIAGRAM OF A POLISHED SILICON STEEL SPECIMEN

As in all fatigue diagrams the stresses are determined by the use of traditional methods, assuming the linear distribution of stress and disregarding the possible stress concentrations. In that respect it is of interest to compare the tension-compression and bending stresses. The comparison between the traditionally assumed and actual stress distribution is shown on Fig. 3(*a*) and Fig. 9 for the tension-compression and bending, respectively. By comparing these two diagrams it can be seen that in the case of tension-compression the actual stress is higher, and in that of bending lower, than the assumed stress. If corrections due to non-linear distribution of stress were applied to the data in Fig. 8, the fatigue strength in bending would be lowered and that in tension-compression would be raised, bringing both of them together. The large difference between the shear and tension-compression fatigue would be made still larger.

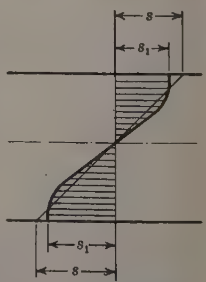


FIG. 9.—DISTRIBUTION OF STRESS IN A RECTANGULAR BAR SUBJECTED TO BENDING.

Comparative Fatigue Diagrams of Different Alloys.—The fatigue strengths of a number of alloys, determined on highly polished specimens under tension-compression stress, are shown in Fig. 10(*a*), based on unpublished tests by R. L. Templin, M. Am. Soc. C. E., and on V. D. I.

publications. The alloys were chosen to represent high-strength alloys of their respective classes.

The ultimate tensile strength and elongation of these alloys are given in Table 1. Although it gives the absolute fatigue strength values, Fig. 10(*a*)

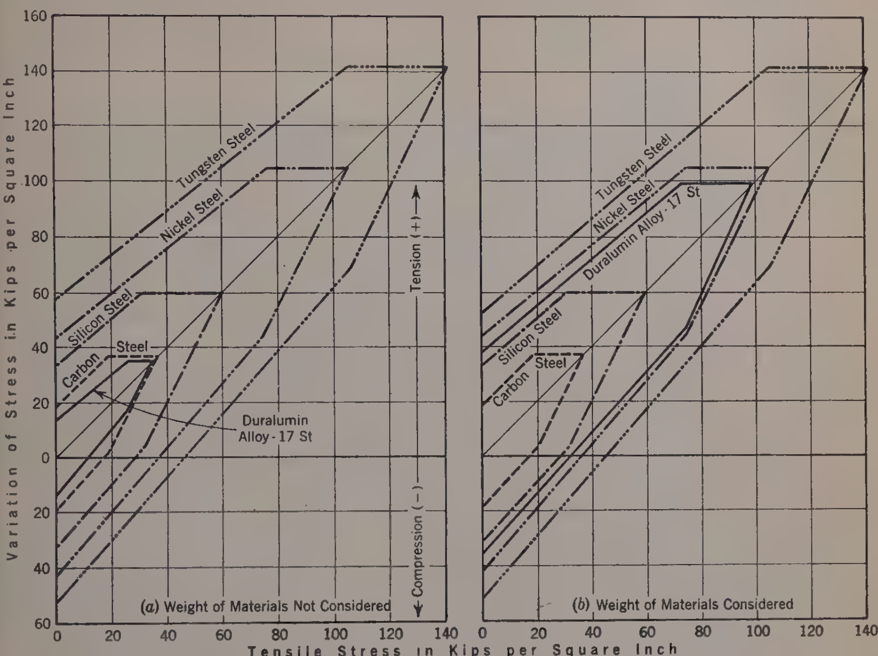


FIG. 10.—COMPARATIVE FATIGUE DIAGRAMS FOR POLISHED SPECIMENS.

does not offer a proper basis for comparison, since the weights of the materials are not considered. Fig. 10(*b*) is based on the same data, but the weight is considered by assuming the steel as the material of unit weight, and multiplying the fatigue stress for duralumin by the steel-duralumin weight ratio, which makes possible a direct comparison of the different alloys on the strength-weight basis.

TABLE 1.—CHARACTERISTICS OF ALLOYS IN FIG. 10.

Alloy	Ultimate tensile strength, in kips per square inch	Percentage elongation, in 4 inches
Duralumin	59	19
Carbon	64	20
Silicon	107	10
Nickel	139	10
Tungsten	170	10

IMPROVED DESIGN METHODS

Determination of Stress.—The assumption that the materials follow Hooke's law does not take care of the individual properties of the different materials,

but bases all designs on the use of non-existing ideal materials, with perfect elastic properties. The assumption of linear stress distribution in many cases precludes the proper determination of stress, particularly at the boundaries.

Considerable experimental and theoretical work was done in the past and is being done at present in order to create a better understanding of the stress phenomena, so that more refined designs may be used in which the individual properties of the different materials are considered and the stresses are determined more exactly.

This work is a part of the widespread experimental and theoretical investigations made during the last few decades and is prompted mostly by the needs of automobile and airplane designers.

The present situation in engineering design is such that the conventional design theories do not supply any methods for determining the exact stress; but in most cases the average stresses can be determined. The maximum stress values which govern the design, and which are of such importance under repeated load conditions, cannot be determined.

The impossibility of developing a strict theoretical method of exact stress determination, and the desire to develop the results of the advanced studies in such shape that they could be used without any difficulty by the practical engineer, resulted in the evolution of combined design methods in which the conventional theoretical methods are utilized, and the data thus obtained are corrected by the use of empirical coefficients.

How far it is necessary to go in applying such methods depends on the exactness with which the design should be made. In aircraft, where the saving of weight governs the design, the most exact application of all available methods is of importance. In structural applications such exactness may not be necessary, but the use of more expensive or less known materials will make advisable the application of methods of greater exactness than the customary methods.

The exact methods that can be applied at present may be divided briefly in four steps:

First.—Assuming that the structure or its elements are built of an ideal material (which is practically non-existent), the stresses are determined by the use of conventional design methods;

Second.—The conventional stresses thus determined, are corrected to bring about close agreement between the design and correct stresses in the ideal material;

Third.—Stresses determined for the ideal material are corrected again, to take care of the substitution of materials actually used for the ideal material, so that a close agreement is assured between the design and actual stresses; and,

Fourth.—The finally determined stresses for the materials that are actually used are compared with the stress-resisting capacity of these materials under the expected loading conditions.

The traditional structural designs are based on the use of the first step only. The fourth step is seldom fulfilled; since the stress-resisting capacity

of the materials is not correlated with the expected loading conditions, in particular, the fatigue properties are usually neglected.

The inclusion of the second and fourth steps in the design will insure much greater precision. All four steps probably represent the most exact design possible at present.

Stability.—Stability problems are of considerable interest in the more refined designs. In the past the tendency has been to design structures so that no stability failures were possible, the failure by overstressing the material occurring under smaller loads than those at which the condition of instability could be reached. In other words, the attempt was to keep the safety factor with reference to stability greater than the safety factor with reference to stress failures.

An economical design not only must be balanced in so far as the stress safety factors are concerned, but it should also be balanced in so far as the stability safety factor is concerned. Theoretically, there are no reasons why a structure or its elements should have different safety factors with reference to stress and to stability. Practically, the stress-carrying capacities of alloys are better understood than the conditions of instability, and the tendency of the designer is to use higher safety factors with respect to stability as compared to stress, with the exception of some aircraft designs.

No matter what the material, and how stiff the structure or each of its elements may be, the strains and resulting deformations and deflections will appear every time forces are applied, the only difference being in the amount of deformations and deflections. These factors will depend on the modulus of elasticity of the material used and the stresses developed.

The mathematical treatment of the stability problem is even less definite than that of other stress problems. The fact that the change from stable to unstable conditions in most cases occurs during a very small change in the loading conditions, makes the exact mathematical solution a problem of such refinement that it cannot be obtained exactly by the use of present mathematical methods.

Practically nothing is known about the question as to whether, and, if so, how, the local stress concentrations and repeated load applications influence the stability. In comparing the application of different alloys, the stability may become an important factor.

If higher stressed, stronger alloys of the same specific gravity and modulus of elasticity are substituted for low-strength alloys, the thickness of the members is reduced, the deformations and deflections are increased, and, consequently, the stability safety factor is lowered.

If alloys, equally stressed and of the same strength but of lower specific gravity and lower moduli of elasticity, are substituted, no direct conclusion may be drawn. The comparative stability will depend on the details of design with respect to the distribution of the additional volume of the alloy of lower specific gravity.

Application to Actual Designs

Necessarily, the refinements of engineering design must start with a more exact prediction of the future loading conditions. Not only should the maximum force to be applied be known but, for varying forces, the amount of variation and the probable frequency of different load application must also be known. All these conditions have an important bearing on the behavior of the structure.

The relation between the shop and field labor costs and between the cost of material and labor are changing factors the importance of which is considerably accentuated since more expensive materials and different jointing methods are put at the disposal of the structural engineer.

Considering the different designs, the fatigue and stress concentration phenomena are of particular interest. The load that a metal structure carries produces stresses that may vary from a perfectly static to a completely reversing stress. There are very few metal structures the members of which are subject to perfectly static stress. Even the steel skeletons of buildings are subjected to a constantly varying stress, due not only to the wind action but also to temperature variations. As a general rule, these stress variations are small as compared with the mean stress. On the other hand, the number of structural elements that are subject to complete stress reversal is also comparatively small. The vast number of structural elements are subjected to varying stresses of different degrees of pulsation or reversal.

The number of stress pulsations or reversals may be considerable. The most outstanding instances are structures that are subject to vibrations in which the number of stress pulsations or reversals may be comparable to the number of stress reversals to which parts of fast running engines are subjected. Most of the structures are subjected to fewer stress pulsations or reversal.

These considerations would indicate that stress concentration and fatigue phenomena must be taken into account in structural design.

Joints.—The necessity to develop expensive high-speed internal combustion engines forced the mechanical engineer to take into consideration and to begin a study of stress concentration and fatigue phenomena. Joints, being the most expensive and most sensitive part of a structural design, are at present forcing the structural engineer to follow the same path.

Riveted joints are necessarily producing stress concentrations. If two similar riveted joints are compared, the traditionally assumed strength of which is the same, it would seem that the joint in which a small number of large rivets is used will have higher stress concentration than one in which there is a larger number of small rivets. All other conditions being equal the joint with a larger number of smaller rivets should have the higher fatigue strength.

The next conclusion would be that a continuous welded joint should have a greater fatigue strength than a riveted joint, which may not always be the case. The local heat application will result in residual stresses that may

increase the peak stress. The different properties of the base material and of the welded material, and the possible changes in the properties of the base material adoining the weld may produce internal boundaries, resulting in high stress concentration similar to those produced by abrupt changes in shape. The stress concentrations thus produced, and the changes in the properties of the base material, may result in a considerably lower fatigue strength of the joint.

Safety Factors and Working Stress.—The adequacy of an engineering structure is expressed in terms of the safety factor, which is supposed to represent the ratio between the actual and the ultimate conditions, under which the structure will fail. The safety of a design is not determined by an average safety factor, but by the lowest safety factor at the danger spot of the weakest element. It may be influenced by the fact that in any redundant construction, the members can partly unload this burden upon other parts of the structure, thus making the actual safety factor higher than the minimum safety factor of a single element. The factor is to be considered first in strength, represented by the stress safety factor, and second in stability, represented by the load safety factor. In the consideration of any given safety factor of a structure the most important item is the reliability of its determination. If crude design methods are used, the actual safety factor in many places or instances may be much lower than the theoretical safety factor. If more refined design methods are used, the actual and theoretical safety factors will be in closer accord.

In practically all cases a structure designed by the use of conventional methods, with a high safety factor, will be less safe than one designed by the use of more refined methods and using a smaller safety factor. The conventional safety factor idea is that the safety factor for a particular design under given loading conditions is a constant coefficient; but actually the safety factor varies.

In all engineering structures the factor of safety decreases as time goes on. In some instances it may initially increase for some time, if the stress-bearing capacity of the material increases, until it reaches its maximum, and it will then decrease. Some of the alloys as well as concrete are typical in that respect. In most structures the initial safety factor is the largest and after the structure goes into service the safety factor decreases gradually. Taking into consideration only the decreasing stress-carrying capacity with repeated loadings, the initial or maximum safety factor in many cases may be twice as high as that which will obtain after a definite time period. The difference will be still greater if the changes in material—its corrosion or deterioration—are considered.

If these conditions are realized it should be clear that a simple statement that the structure is designed with a certain safety factor is not sufficient. The safety factor should be qualified. In structural designs in which the fatigue properties and the stress concentrations are neglected, the design safety factor represents a theoretical initial safety factor, which actually is not attained even at the initial period.

The safety factor specified in modern aviation designs in which the fatigue properties and stress concentrations are considered, represents the actual safety factor which the structure will have at the end of its useful life, the initial safety factor being higher. The selection of the proper safety factor requires considerable engineering judgment. Too high a factor will result not only in an uneconomical structure, but will make it difficult (as, for instance, in long span bridges) or even impossible (as in aviation), to design a structure that could fulfill its functions. The danger of too low a safety factor is obvious. In practical applications the safety factors vary between 1.25 and 4.0 and often are even higher. It seems only reasonable to accept lower safety factors for more refined designs and to accept high factors for less refined designs in which only the initial theoretical safety factor is considered.

Region of Permissible Stress Variations.—The data given in a 45° diagram may be applied to a design if the proper reductions are applied. Fig. 11 shows the method of applying reductions. It is based on data given in the previous diagrams on tests made on highly polished small-sized specimens.

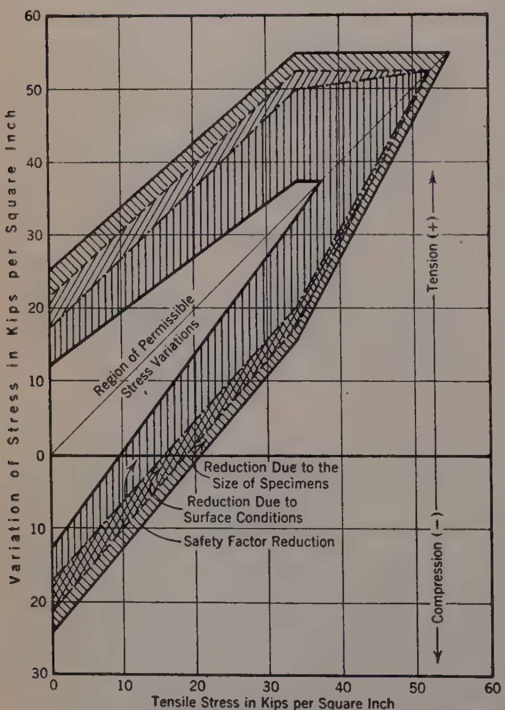


FIG. 11.—FATIGUE DIAGRAM FOR POLISHED SPECIMENS.

Part of the reductions must be applied to the stress variations. The first reduction applied takes care of the difference between the size of the testing specimen and the actual structural elements, and the possible difference in their properties. A small reduction in stress variations should take care of the usual difference in sizes. An increased reduction would be necessary if there is a difference in properties of the test specimens and the material actually used.

The next reduction shown in Fig. 11 should take care of the difference in surface conditions of the test specimens and the actual element. This reduction should be again applied to stress variations and should be taken from a diagram similar to the one shown on Fig. 5, giving, for instance, the reduction between highly polished and machined specimens.

The final reduction will be the safety factor reduction, which should be applied to the maximum value of stress left after the previous reductions were applied. The area finally left, shown in Fig. 11, will give the region of permissible stress variations, which should govern the design.

Conclusion

During the last few decades the theoretical bases of Structural Engineering have been very much standardized and have changed slowly. Structural engineers used ordinary mild steel and there was no particular need of refinement in material or in methods of design. At present, there seems to be a gradual change in conditions, making advisable a more critical attitude.

Conventional designs neglect stress concentrations, fatigue, and creep phenomena, and assume perfectly elastic properties of the materials. Such designs are idealized, presuming a mode of load application and jointing (which in the majority of cases may not be approached), and the use of non-existing ideal materials.

It is only natural that the development of engineering science should have started with the simplified, idealized conditions, and that at present the trend is toward evaluating the adjustments that are necessary in order to force the idealized and the actual conditions into a better agreement.

A clear visualization of these conditions will make possible the determination of the degree of refinement in the theoretical design that is practical and reasonable. There is a widespread belief that better designs are obtained if the conventional theories are applied and extended to cover a number of possible secondary influences.

Considerable time may be spent and very elaborate theoretical values obtained, which may represent a very refined extension of the conventional theories, but in most cases such efforts do not disclose the behavior of the actual structure that is not subjected to idealized loadings and is not made of an idealized material.

The conventional theories being only an approximation, the further extension of such approximate theories will result, in many cases, in larger discrepancies between the evaluated and actual behavior of the structure. More refined designs will involve additional engineering work, and it is desirable at least to indicate the methods by use of which the additional time invested in a design will bring returns in assuring a closer agreement between the evaluated and actual conditions. This purpose may be stated as the designing of structures in which the evaluated and the actual safety factors are identical.

The paper records an attempt to outline broadly the factors that should be taken into account. In the following papers of the Symposium some of the points mentioned in this paper are developed more authoritatively and in more detail. It is believed that at present the developments are too fluid to be stated in terms of definite rules and recommendations, although there is a possibility that the static conditions of the past will not return for many years. In that event, the structural engineer must change his attitude in line with that of the more advanced branches of engineering.

TESTS OF ENGINEERING STRUCTURES AND THEIR MODELS

By R. L. Templin[4], M. Am. Soc. C. E.

Synopsis

The scope of this paper is limited to tests of structural members, involving the determination of deflections, stresses, and general behavior under given loads, of engineering structures and their models. Following mention of the purposes and types of the tests made, is a discussion of the details of the methods used. Consideration is given to similarity conditions, model materials, testing apparatus, and testing technique. Reference is made to specific examples of tests of actual structures, and of small-sized and over-sized models. The results from an over-sized model example are given, illustrating the various purposes mentioned.

From consideration of the factors affecting structural tests of engineering structures and their models, and the results obtained from such tests, it is concluded that the purposes enumerated can be fulfilled if proper consideration is given to actual departures from strict similarity conditions.

Types of Engineering Structures and Tests to Be Considered

Within recent years there is evidence of an increase in the number of engineering structures that are tested, either during or after construction, or both, under definitely imposed conditions of loading. Although these tests have been made with some differences in purpose, in general, they have been conducted with the intention of checking the design assumptions, postulates, and calculations. Examples of such tests may be found in the publications of the Society, particularly in the *Transactions* for the past seventeen years (1919–1936). Tests of the kind referred to, frequently afford a very satisfactory accelerated method for revealing the adequacy or inadequacy of the structures in lieu of the more generally used tests of time and service.

The types of structures that may be tested include essentially all those made of engineering materials with apparently no serious limitations being imposed by size, use, or location. The scope of this paper, however, will be restricted to structural tests involving determination of deflections, stresses, and general behavior under given loads.

The designs of most modern engineering structures are based on: (1) Some given over-all dimensions; (2) certain physical properties of the materials involved; (3) assumed conditions of loading; (4) analyses of stresses, deflections, and stability; (5) empirical rules; (6) "engineering judgment" with

[4] Chf. Engr. of Tests, Aluminum Co. of America, New Kensington, Pa.

"suitable" allowances for safety; and usually with (7) consideration of the economics involved.

When an engineering structure performs the expected duties throughout or beyond its estimated life, it is generally considered a successful engineering project. Most engineering structures have successfully fulfilled their designers' intentions, but this does not mean, necessarily, that the actual behavior of the structures under service conditions, or the magnitude of the stresses throughout the various parts of the structures, have been in close agreement with the assumed or calculated values. Although many engineers are familiar with the assumptions involved in the design of a given structure, there seems to be a lack of data to show quantitatively the discrepancies between the actual behavior either of the structure as a whole or of its various parts and the predetermined theoretical values. The quantitative effects of the errors involved in the usual assumptions made concerning isotropy and homogeneity of materials, Hooke's law, continuity, and fixity are often difficult to evaluate.

In order that a better understanding of these factors, as well as of many other factors involved in the behavior of some engineering structures, either under arbitrarily imposed or actual service conditions, may be had, tests have been made on actual structures or on their models. In the majority of cases the results obtained from such tests have indicated quite definitely that this field of endeavor might be considerably enlarged with consequent advantages to all concerned Among the various advantages that might be anticipated, would be more efficient, safer, and, probably, more economical structures.

Purpose of Tests.—Tests of actual structures are usually made for any or all of the following purposes: (1) Checking the analysis against the actual behavior of the structure under known load conditions; (2) checking the actual behavior of the structure under service conditions; (3) providing data to be used as a basis for changes in design rules; and (4) checking the efficacy of any alterations made in an existing structure.

Tests of models of engineering structures may be made for any or all of the following purposes: (5) To check analysis by actual measurement of the behavior of the model under known load conditions; (6) to provide data to be used as a basis for changes in design rules; (7) to check the efficacy of any proposed alterations in a given construction; (8) to supplement theoretical analysis by experimental analysis; (9) to avoid theoretical analysis by experimental analysis; (10) to provide an easier, quicker, and usually a less expensive means of obtaining the desired information, than would be the case if the actual structure were tested; and (11) to provide a means for studying the behavior of a design under loading conditions not possible with a full-sized structure.

Types of Tests.—Types of mechanical tests which may be conducted with the foregoing purposes in mind, may be subdivided into two general classes: (*a*) Static; and (*b*) dynamic.

Under static tests may be included the determination of strains (and, from these, stresses) and deflections, at critical points or throughout the structure, resulting from known conditions of loading.

Dynamic tests include those involving the measurement of strains (and, hence, stresses) and deflections resulting from the impact of moving loads, as well as the determination of the natural vibration frequencies of various parts of the structure, or of the structure as a whole.

In both types of tests it should be emphasized that the stress and deflection values obtained, represent changes resulting from the conditions imposed, and although corrected for "no-load" or "dead-load" or "at-rest" conditions, should not be designated as "true" or "absolute" stress or deflection. Even a limited understanding of the strains induced in materials, especially metals, during production, and in the actual structure by fabrication and erection, would mitigate seriously against the use of such appellations unless heterodox definitions of their meaning are to be accepted.

Test Methods

Size of Models and Materials Required.—Considerable discussion has appeared in technical literature concerning the laws of similarity to be followed when using models to analyze the behavior of engineering structures. The papers by B. F. Groat[5], M. Am. Soc. C. E., and the discussions pertaining to them, have emphasized the theoretical requirements for models which are intended to comply with strict similarity conditions. In these same discussions it has been shown that one may make some departures from the ideal conditions and yet obtain worth while results. The writer is quite in agreement with the latter idea after a number of years of experience in the use of models to assist in analyzing engineering structures. Although it is recognized that extensive tests of actual-sized structures are preferable in many instances, for obvious reasons, they are often impracticable and, in some instances, impossible. Supplementary partial tests on actual structures frequently afford a satisfactory final check on both theoretical and experimental analytical studies, as, for example, those made on the towers of the George Washington Bridge[6].

Models of engineering structures are generally thought of as being smaller than their prototypes. This need not always be the case, however. In some instances over-sized models can be, and have been, used to advantage. An example of tests on an over-sized model will be cited subsequently. Irrespective of whether the model size chosen is larger or smaller than the prototype, conditions of testing, in most cases, can be controlled to better advantage than in the case of tests of actual structures.

Models Made of the Same Material as the Prototype.—As indicated by Mr. Groat[5] an engineering model should be one which is scaled down, or up, in such a way that the dimensional and time requirements explained in his paper are fulfilled. These are theoretically ideal conditions which are very difficult to comply with in actual practice. Consider, for instance, a model of a truss in which structural members are fastened together by means of

[5] "Ice Diversion, Hydraulic Models, and Hydraulic Similarity", *Transactions* Am. Soc. C. E., Vol. LXXXII (1918), p. 1138, and "Theory of Similarity and Models", *Loc. cit.*, Vol. 96 (1932), p. 273.

[6] *Transactions*, Am. Soc. C. E., Vol. 97 (1933), p. 181.

gusset-plates. If the model is a photographic reproduction of the prototype, the cross-sectional areas of all the members will be changed in the proportion of the square of the scale ratio, but the moments of inertia of the members will be changed in a different proportion. The stiffness factors of the members—that is, the moment-of-inertia-to-length ratios of the various members—however, will be changed to still another scale. The stiffness of the gusset-plates will be changed to a scale different from that of the cross-sectional areas. Consequently, when the various members are considered individually, the degree of end restraint will not be the same as in the prototype, and the behavior of the various members of the model, under combined axial loads and bending, will be different from that of the corresponding members of the prototype.

In the laboratories of the Aluminum Company of America, at New Kensington, Pa., the procedure of testing models of structures has been to consider the model itself as a structure and to make a combined analytical and experimental study of it as such, to determine its behavior under various types of loading. By so doing, all the factors introduced are treated automatically in the light of their proper magnitude, so that when the prototype is being studied, the various elements entering into the behavior of the structure are not considered by simply applying a scale factor to the results of the model tests, but the various factors and considerations are included in a detailed analysis. In joining various members to make an assembled structure, it is impossible to reproduce, in a small or in a large model, the same conditions that are normally obtained in the actual construction of its prototype. Therefore, the effect of connections cannot be determined reliably from small or large models unless all the factors entering into the determination are known. This is especially true in connection with loads which normally will produce fairly high stresses in the prototype and may cause a local yielding of the material, whereas, in a small model of the structure, in which the stresses are scaled down automatically, this local yielding does not occur.

In many models which depend for load upon their own dead weight, or on the weight contained within them, it follows inevitably that if a direct scale reproduction of the prototype is made, the resultant stresses in the various parts of the model structure will be changed also, to the same scale ratio. This procedure makes any measure of strength obtained from models of this type very ineffective, unless analytical work involving all the variable factors is carried on and used as a basis for interpreting test data.

In many structures the ultimate safe load that can be supported depends upon the stability of the structure as a whole, or of some of its parts. In general, the stress at which a particular member becomes unstable, is a function of the proportions of its parts; as, for instance, the stress at which a thin outstanding plate buckles is directly proportional to the modulus times the square of the ratio of thickness to the outstanding width. Thus, the stress at which an outstanding flange would become unstable would be the same for a prototype or a model, with the result that if the stresses in the model are changed as the scale ratio, a false sense of stability is obtained.

Models Made of Materials Different from Those of the Prototype.—It may be desirable to make models of an engineering structure from a material different from that of the prototype, so as to:

(1) Correct for the stresses that actually occur in the model; that is, the ultimate strength of the material for the model should be changed to the same proportion of the actual stress under test as the ultimate strength of the prototype bears to its actual stress in service.

(2) Correct for the stability; that is, to have a material with a modulus sufficiently low, so that the stress at which the various component parts will become unstable bears the same relationship to the stresses in the model, as the stress at which the prototype becomes unstable bears to the actual stresses in it.

(3) Correct for the different relative distribution of direct stresses, bending, and shear, by having either auxiliary loads or a composite model made up of two materials so that resistance to shear and bending of the model will be the same as that in the prototype.

(4) Correct for the relative proportion of dead and live weight in the model and in the prototype. In this event the specific gravity of the model should be comparable to the specific gravity of the prototype.

(5) Provide the same distribution of stress in two-dimensional or three-dimensional models. This requires the Poisson ratios for the materials in the model and the prototype to be equal.

In order to interpret model studies, Factors (1) to (5) must be considered in connection with the theoretical similarity studies indicated in Mr. Groat's papers previously cited, because, if any one of the conditions enumerated is violated, it may be that a direct comparison cannot be made between the model and the structure. Since it is practically impossible to fulfill all these conditions simultaneously, it becomes necessary to supplement model studies with adequate theoretical analyses in order to take into account the effects of the various factors.

In selecting the size of model to be used, consideration should be given to the size of the testing apparatus available and the magnitude of the strains and deformations to be measured. Even if specially designed apparatus must be used, for any given tests, limitations will be imposed, which must be recognized if the desired results are anticipated.

Testing Apparatus.—In the testing of actual structures or models it is customary to apply either static or dynamic loads, or both. In the case of static loads applied to actual structures, dead weight of one form or another is frequently used. Almost any form of available material, of suitable density, may be used for such loading, but care should always be taken to insure that the load is applied to the structure in as close agreement as possible with the assumed conditions of application, or in accordance with service conditions (preferably both), if close agreement between calculated and actual values is anticipated. The method sometimes used for loading both actual structures and models, is a cause of a considerable number of the discrepancies observed. There are numerous methods, other than dead

weight, for applying loads to the structure being tested; for example, calibrated hydraulic jacks or springs can often be used to advantage. In the case of laboratory tests, some arrangement such as that used in the Materials Testing Laboratory of the University of Illinois[7] is worthy of comment. A part ($23\frac{1}{2}$ ft by 120 ft), of the concrete floor of the laboratory is heavily reinforced and provided with suitable inserts at regular intervals, into which may be screwed tension bolts that furnish the necessary reactions for any structure to be tested. When the structure is restrained by this method it may be loaded with dead weights and levers, or hydraulic jacks or springs with load-indicating devices.

When the structure to be tested can be put in a testing machine, the loading problems are usually very much simplified, although under such condition restraints may be applied to the structure under test, that are not in strict accordance with the assumed or desired loading conditions. The torsional and bending resistances offered by testing machines in the test of columns, using any type of heads except those substantially free from friction, is an example. The spherical heads usually provided with commercial testing machines are not substantially free from friction while under load. Furthermore, considerable lateral restraint is imparted to beams by testing machines when using the testing procedure generally followed, and this lateral restraint may appreciably affect the test values obtained.

Changes in the strains occurring in the various parts of a structure may be determined by using any one or more of a variety of strain-gages. Although a number of such instruments of different sizes and types are available commercially, special designs are evolved, occasionally, to meet particular requirements. The types used may be classified as mechanical, electrical, optical, acoustical, or some combination of the four. Some of the instruments are intended to be applied manually to the structure while making an observation, and then removed. Others are attached permanently for the duration of the test. Both indicating and recording (intermittent and continuous) types are available. Some can be read locally only, whereas others permit of distant observation.

The instruments to be selected for field use, in most instances, should be more rugged than those which may be used in the laboratory. Increased ruggedness of design, unfortunately is frequently accomplished at a sacrifice of sensitivity or accuracy, or both. Temperature effects are often negligible under laboratory conditions of test, but must be considered carefully in the field; thus, certain parts of the better field types of strain-gages are made of invar. Even so, such types of instruments should be checked with suitable standard bars to correct for residual temperature effects as well as for incidental maladjustments. An example of the consideration which should be given to a selection of strain-gages may be found in the report of the Committee on Arch Dam Investigation[8].

---

[7] Described in pamphlet issued by Univ., of Illinois for distribution at the dedication of the Laboratory, May 2, 1930.

[8] *Proceedings*, Am. Soc. C. E., Vol. LIV, Pt. 3, May, 1928, p. 64.

Regardless of the type of strain-gages selected, considerable attention should be given to the details of the methods of applying the instruments to the gage lines, throughout which the changes in strain are to be measured. For those strain-gages which require holes at the ends of the gage length, variations in the size and condition of the edges of the holes used affect both the accuracy and the sensitivity of certain instruments more than others, depending on the mechanical design of the instrument. When clamps or machine screws are used for attaching the instruments, appreciable errors may occur because of an indeterminate degree of restraint in the apparatus. The method of attachment sometimes used with some types of tensometers, or strain-gages, are examples.

The sensitivity, accuracy, and total range of available strain-gages vary appreciably, and, therefore, deserve careful consideration from the viewpoint of what is desired in any given test. Just because a strain-gage is sensitive to $\pm$ 0.00001 in. per in., it should not be inferred that the instrument is accurate to the same degree. Quite frequently the limit of accuracy will be found to range from three to thirty times this value. The total range of an instrument may become a limiting factor when attempting to measure large strains. Some of the instruments, notably those of the optical type, have very limited total ranges or give values for large deformations which may include appreciable errors that would be negligible in the case of small deformations. There are a number of other details which should be given consideration in the application of strain-gages to a structure, which will be dependent on the particular instrument used, as well as on the test conditions. Only a few of the more important factors involved have been mentioned, for the purpose of emphasizing the need for attention to such details if the best results are desired.

Measurements of the deflections of structures, or of their component parts, require first a definition of the desired quantities with respect to some given datum planes or bench-marks. Under test conditions, the reference planes or marks should remain fixed or, if they do move, both the magnitude and direction of such movement should be known. Deflections of a beam, for example, are best determined by reference to points on the neutral axis directly over the centers of the supports, and deflections of a pin-connected truss by reference to centers of the end pins at the supports. Deflections of a concrete arch dam, however, might be construed as including deflections of the arch proper, the movement of abutments and of that part of the cañyon affected by the load on the structure, with reference to fixed points beyond the influence of the structure and its load. There are excellent opportunities for both confusion and errors, in attempting to interpret given deflection values for structures, where there is evidence of lack of appreciation of the requirements for proper reference planes or points

The instruments for deflection measurements are so varied in character and type as to preclude any discussion other than mere mention of a few of those more commonly used: Scales, micrometer screws, dial-gages, tapes, plumb-lines, levels, theodolites, and combinations of some of these devices. Likewise, their sensitivity and accuracy vary over wide ranges. It will

suffice for the purposes of this paper, to emphasize the necessity for selecting suitable deflection-measuring apparatus with regard to the character of the values to be determined and the desired accuracy, rather than let the choice depend entirely on availability of equipment. Deflection measurements, when properly made and correlated with known loading conditions, may, and generally do, give reliable data on the behavior of a structure and serve as a valuable check on the theoretical or experimental analysis.

All the measuring devices used in testing structures should be calibrated by comparison with acceptable standards, preferably those furnished by the National Bureau of Standards, U. S. Department of Commerce, Washington, D. C. Many types of measuring instruments, such as strain-gages, change with use because of wear in the moving parts or accidental misuse. In such instances, additional calibrations should be made at intervals, depending on the specific instrument and the usage it has received. Calibration devices suitable for checking load, strain, and deflection-measuring apparatus, are available, and can be certified as to accuracy by the Bureau of Standards, and then used by any one possessing them, for calibrating the instruments for which they are suited. The degree of accuracy of measurements, suggested by so much attention to calibration of apparatus, may not be in harmony with some of the tests of structures heretofore made; but in conformance with the idea of attempting to enhance, materially, the value of experimental analyses of structures, it is believed that more attention should be given to details of calibration of the instruments used.

Testing Technique.—In the testing of structures, the errors arising from the personal equations involved in the manipulation of the instruments used, must be considered, in addition to those already mentioned. The magnitudes of these errors usually become less, at a diminishing rate, as the experience of the observer increases. Some data exemplifying the magnitude of the personal errors, involved in strain-gage measurements, are shown in Fig. 12. The ordinates of these curves represent the differences in the lengths of the gage lines determined from two readings, and the abscissas represent the percentage of the total number of gage lines on which differences in readings did not exceed values indicated by ordinates. The data were selected at random from a large mass of similar data available. Fig. 12(*a*) shows a comparison of data obtained by two different observers using the same 2-in. strain-gage, on gage lines located on horizontal, vertical, and oblique surfaces. The data shown represent measurements on 536 gage lines by Observer *A* and on 569 gage lines by Observer *B*. In Fig. 12(*b*), the group of readings by Observer *B* has been broken down with respect to position in order to show how that factor affects the personal equation involved. Fig. 12(*c*) shows a comparison of results obtained by two different observers, one using a 2-in. strain-gage and the other a 10-in. strain-gage of a different type. The data given are from measurements taken on 1 050 gage lines by Observer *C* and on 310 gage lines by Observer *D*.

Observed values obtained during the tests of structures, must be corrected frequently or adjusted to compensate for changes occurring in the test conditions, such as rise or fall in temperature. Just how to make corrections

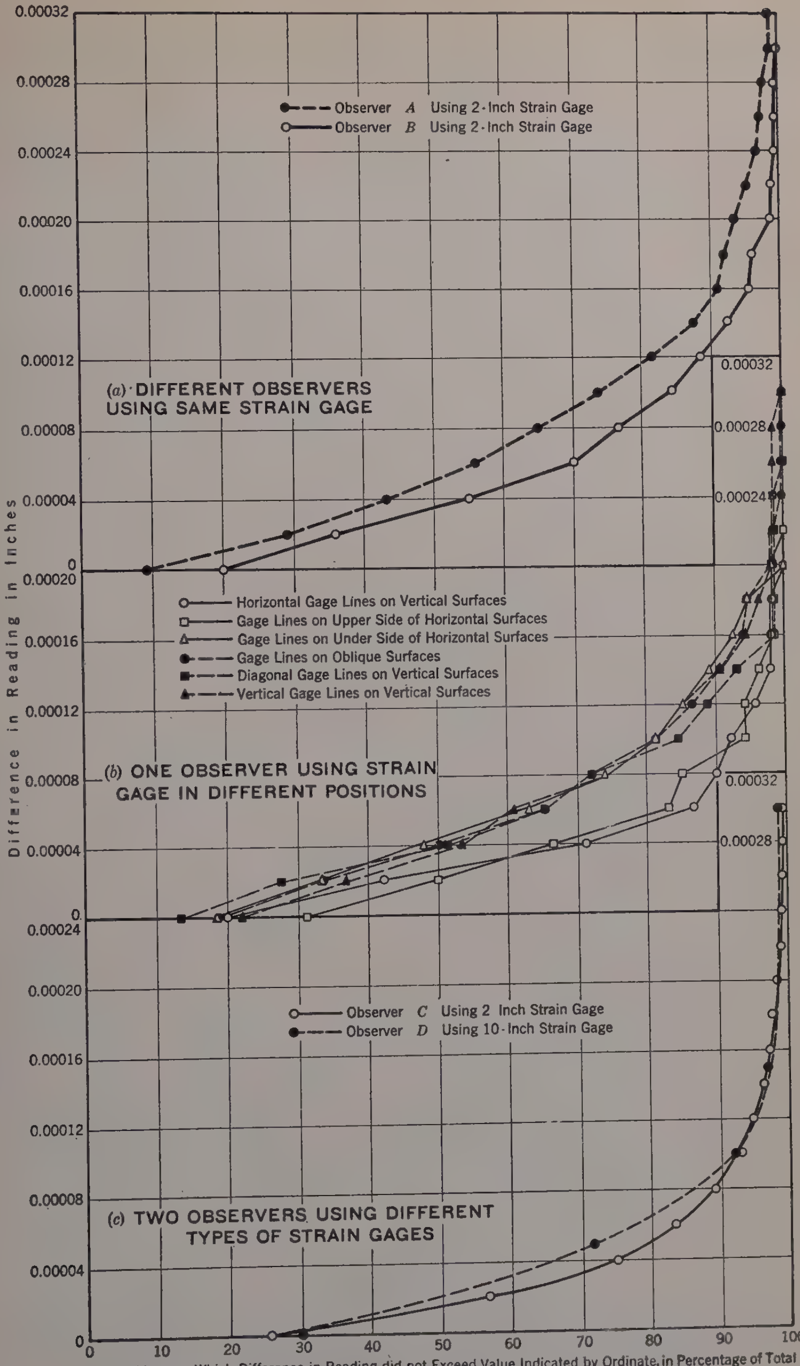


FIG. 12.—MAGNITUDE OF PERSONAL ERRORS INVOLVED IN STRAIN-GAGE MEASUREMENTS.

for these changes, often presents problems that are not susceptible of accurate solutions. In the case of strain measurements, it is rather common practice to use reference specimens of the same material as the structure, similarly exposed to the same elements causing change in temperature, so that the changes in the gage lines on the specimens will be caused only by temperature, time, and the coefficient of expansion of the material. The observations made on these reference specimens are used to adjust the measurements made on the structure, usually on the assumption that there has been a linear change in temperature between times of observations on the reference specimens. Such a procedure may introduce errors of magnitude which will depend on the discrepancies existing between actual and assumed conditions. Another procedure is to attempt to select a location on the structure being tested, at which there will be no stress induced by the loads imposed, and yet where temperature changes similar to those in the remainder of the structure will occur. At this location a gage line may be placed which will serve as a means for obtaining temperature correction values. It is often difficult, however, to select a location where there would be no doubt about the absence of strain, resulting from the loads imposed on the structure. Still another method involves the use of invar reference bars for checking the instruments used, temperature corrections being calculated from the measured temperature changes and the coefficient of thermal expansion of the materials. Notwithstanding the difficulties presented by temperature changes beyond control, experience has shown that in most instances the test data can be reasonably adjusted to compensate for these effects and give quite satisfactory results.

In selecting the size of strain-gage for use in tests of a structure, consideration should be given to the radius of curvature of the surfaces on which the gage lines will be located, in the plane of the gage line. If the radius of curvature changes appreciably during successive observations, apparent strain values will be obtained, which cannot be reduced to stresses by the usual procedure of multiplying strain by modulus of elasticity. Measurements taken on thin plates where local buckling occurs, may indicate, erroneously, exceptionally high or low stresses, depending on the change in radius and direction of curvature of the plate. The use of strain-gage readings at any given location, on opposite surfaces, is generally necessary in order to obtain a correct picture of the stresses at that location. Furthermore, when strains are to be measured in regions of high concentration of stress, or where steep stress gradients occur, it is desirable to use as short a gage length as possible so that the magnitude of the stresses may be determined within satisfactory limits. These instances serve to emphasize the point that a selection of a suitable size of strain-gage (or gage length), on the basis of the sensitivity and accuracy of the instrument itself, does not always insure accurate values for stress measurements.

Specific Examples

To illustrate some of the points raised in this paper, reference will be made to a number of specific examples illustrating cases where the various types of information previously listed have been obtained.

Actual Structures.—The tests made on the Hell Gate Bridge[9] comprise an example of those made for the purpose of checking analysis by measurements on the actual structure. The field tests referred to in the reports of the Special Committee on Stresses in Railroad Track[10] and of the Special Committee on Impact in Highway Bridges[11]; and the tests made on steel railway bridges[12] are examples of checking the actual behavior of structures under service conditions. The tests made on Stevenson Creek Dam[13] were instituted for the purpose of providing new experimental knowledge that would assist in better design rules for such structures. Extensive strain and deflection measurements, made on the Santeetlah pipe line[14] afforded a very satisfactory check on the efficacy of alterations made in the design of stiffener rings and supports after partial collapse of the structure during its initial use.

Model Tests.—Tests made on the model of Boulder Dam[15] are an example of those made to check the analysis by actual measurements of the behavior of a model under given load conditions. A series of models of pipe-line designs were made in connection with the investigation of the Santeetlah pipe line[14] for the purpose of providing data to be used as a basis for changes in the design rules.

One of the purposes of the tests by Wilbur M. Wilson[16], M. Am. Soc. C. E., on multiple-span concrete arches with and without decks, was to check the efficacy of variations in a given type of construction. The model tests in connection with the George Washington Bridge towers[17], were made to supplement theoretical analysis by experimental analysis. The model tests of multiple-span arches conducted by George E. Beggs[18], M. Am. Soc. C. E., offered a means of avoiding theoretical analysis by experimental analysis. Substantially all the tests on model dams are examples of efforts made to provide easier, quicker, and usually less expensive means of obtaining the desired information than would be the case if the actual structures were tested. Model tests made for the purpose of predicting the behavior of a falling dam[19] afford a good example wherein the use of models was a means for studying the behavior of a design under loading conditions not possible with the full-sized structure.

Numerous examples of models made from materials different from those used in their prototypes, may be found upon reference to the technical literature. A single example only, has been selected to exemplify each of the five reasons for using different materials, previously cited. Because aluminum models were used in the tests made in connection with the investigation of the Santeetlah pipe line[14], it was necessary to correct for the stresses that

---

[9] *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1040.
[10] *Loc. cit.*, p. 1191; and, also, Vol. LXXXIII (1919–1920), p. 1409.
[11] *Loc. cit.*, Vol. 95 (1931), p. 1089.
[12] *Bulletin*, A. R. E. A., Vol. 37, No. 380, October, 1935.
[13] *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3.
[14] *Transactions*, Am. Soc. C. E., Vol. 98 (1933), p. 154.
[15] *Engineering News-Record*, April 7, 1932, Vol. 108, No. 14, p. 494.
[16] *Transactions*, Am. Soc. C. E., Vol. 100 (1935), p. 424.
[17] *Loc. cit.*, Vol. 97 (1933), p. 179.
[18] *Loc. cit.*, Vol. 88 (1925), p. 1208.
[19] *Civil Engineering*, July, 1932, Vol. 11, No. 7, p. 415.

actually occurred in the model. The series of model tests conducted by H. E. Saunders and D. F. Windenburg for determining the strength of thin-walled structures[20] included corrections in the loads applied in order to give the same effective stability in the models as in the prototype. The model tests[21] by the late A. H. Beyer, M. Am. Soc. C. E., and Mr. A. G. Solakian, wherein photo-elastic methods were used to analyze stresses in composite materials, involved a choice of materials to correct for the different relative distributions of stresses, by using composite models of aluminum and bakelite. A special material was developed and used in the model tests of Calderwood Dam[22] for the purpose of correcting for the relative proportion of dead and live load weight in the model and in the prototype. The material used for a model of Boulder Dam[23] was selected, among other reasons, because it had a Poisson's ratio substantially the same as the concrete which would be used in the prototype.

Over-Sized Models Tests.—In each of the foregoing examples of model tests the size of the model was smaller than its prototype. In certain instances, however, more satisfactory results can be obtained by the use of over-sized models. The results from such experimental tests, made on what may be considered an over-sized model of a filleted, re-entrant, 90° corner, will be given as an example of such tests. The reason for using an over-sized model in this particular case was that with such device it was physically possible to obtain the stress distribution with available strain-measuring equipment, whereas, the usual full-sized re-entrant corners are so small as to make strain measurements quite difficult and unsatisfactory. Fig. 13 is a view showing the method used in testing a series of specimens cut from a plate $1\frac{1}{16}$ in. thick. The material used for these specimens was a high-strength aluminum alloy (17S–T)[24] plate (Young's modulus = 10 300 000 lb per sq in.; and the proportional limit = 20 000 lb per sq in.). The specimens were loaded in a testing machine having seven different capacity ranges, vary-

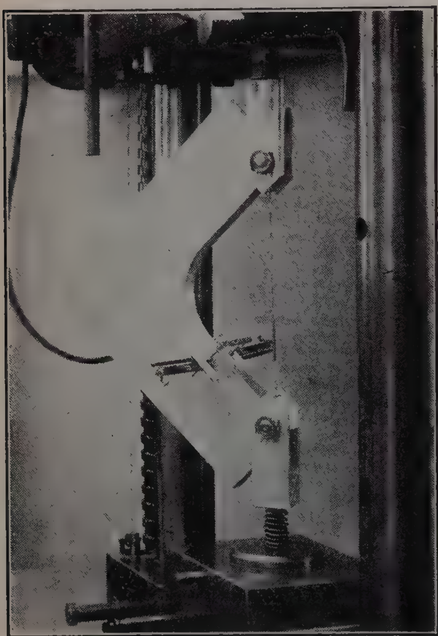


Fig. 13.—View Showing Method Used in Testing 90 Degree, Re-Entrant, Angle Specimens.

[20] *Journal of Applied Mechanics*, A. S. M. E., December 15, 1932, Vol. 54, No. 23, p. 263.

[21] *Transactions*, Am. Soc. C. E., Vol. 99 (1934), p. 1196.

[22] *Loc. cit.*, Vol. 100 (1935), p. 185.

[23] *Loc. cit.*, p. 240.

[24] The symbols in parenthesis denote the trade designation by which the alloy is commonly recognized.

ing from 400 to 40 000 lb, but in this particular series of tests the range chosen was the lowest that would give the desired load. In all cases the load could be determined within ± 1 per cent. The machine was previously calibrated in accordance with Tentative Methods of Verification of Testing Machines of the American Society for Testing Materials[25].

Two tensometers, having 0.5-in. gage lengths and magnification ratios of 2 000:1, were used to determine the strains. The instruments were placed in corresponding positions on opposite faces of the specimen, and the readings were averaged to take care of any variations that occurred throughout the thickness of the specimen. The magnitude of the load applied, in any instance, was that which would give a readily observable maximum strain value but was insufficient to distort the specimen permanently. To insure against permanent distortion and partly to provide measurements of the actual distortion under load, the distance from the line of action of the force to the center of the face of the fillet was measured in each test, using the taut wire shown in Fig. 13, and a suitable scale. In addition, the spread of the angle was measured by means of a 10-in. strain-gage, on a gage line parallel

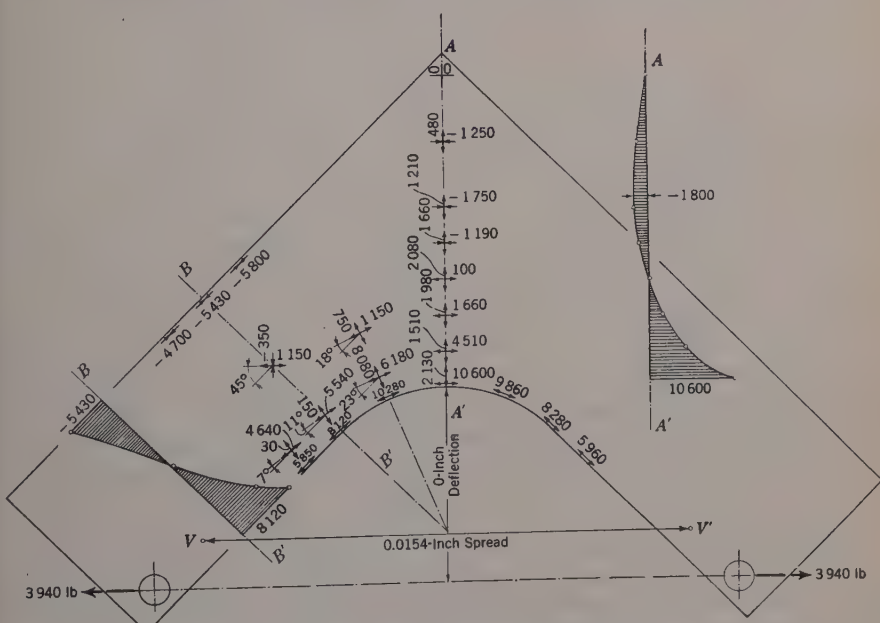


FIG. 14.—DISTRIBUTION AND MAGNITUDE OF STRESSES IN RE-ENTRANT 90-DEGREE ANGLE SPECIMEN HAVING 3-INCH RADIUS FILLET

to the direction of load application. The tensometers and the 10-in. strain-gages were previously calibrated, using a device described elsewhere[26], and found to be correct within ± 0.000015 in. per in.

Observations were made under no-load and full-load conditions throughout most of the tests. Special tests, however, were made for the purpose of

[25] *Proceedings*, A. S. T. M., Vol. 35, 1935, Pt. I, p. 1303.
[26] *Loc. cit.*, Vol. 28, 1928, Pt. II, p. 714.

demonstrating that the load-strain relationship on any given gage line, throughout the range of loads used, remained a linear one, thus making it necessary to determine two points only (no-load and full-load), to define the complete load and strain relationship at any point. The special tests also showed that the distortions of the specimens varied directly with the load.

Fig. 14 shows the magnitude and direction of the stresses obtained from one of the test runs made. The small arrows are on the locations of the various gage lines, with diverging arrow-heads indicating tension along the gage lines and converging arrow-heads indicating compression. Stresses, in pounds per square inch, along the gage lines, are indicated by the numbers directly adjacent to each line. The directions and magnitudes of the stresses at points away from the edges of the specimen were obtained by measuring strains on rosettes[27]. The magnitudes and directions of the stresses at such points were determined by the dyadic circle method.[27]

With these data available on a series of angles of different proportions, a stress concentration factor was determined, as shown in Fig. 15. This

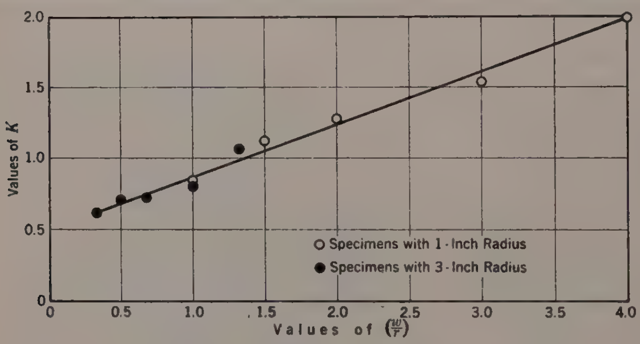


FIG. 15.

stress concentration factor, denoted by K, has been defined as the ratio of the actual measured maximum stress in the fillet, to a nominal stress which may be computed readily. This nominal stress, S_1, is that computed for the inner edge of one leg of the angle at its intersection with the other leg, assuming no fillet and no stress concentration. When values of K were plotted against the ratios of width of specimen to radius of fillet, $\left(\frac{b}{r}\right)$, a straight-line relationship was obtained from which it was found that for the specimens tested, the stress concentration factor could be expressed by the equation,

$$K = 0.5 + 0.37\left(\frac{b}{r}\right) \quad \text{.....................(1)}$$

For other loading conditions different stress concentration factors would be obtained.

[27] *Journal of Research,* National Bureau of Standards, Vol. 10, May, 1933, Paper No. 559, and Vol. 15, December, 1933, Paper RP851.

An experimental study of this same type of problem has been made by means of photo-elastic methods[28]. In this connection, it is interesting to note that with a slight modification of the procedure and technique of strain measurement on over-sized models, it would be possible to extend the stress concentration studies to the case of local bending in the outstanding legs of structural shapes, whereas the photo-elastic methods are definitely limited, at present, to studies in a single plane.

Over-sized model tests were also found very useful in evaluating the strains occurring in multi-stranded cables under conditions of vibration[29]. In this particular instance the experimental analysis was supplemented by theoretical analysis and the two correlated very closely indeed.

Conclusions

From a consideration of the factors affecting structural tests of engineering structures and their models and the results obtained in many instances, from such tests, it may be concluded that:

(1) Tests of actual structures can be made, which will yield satisfactory results when the purposes of such tests are: (*a*) Checking an analysis by observing the actual behavior of the structure under known load conditions; (*b*) checking the actual behavior of the structure under service conditions; (*c*) providing data to be used as a basis for changes in design rules; and (*d*) checking the efficacy of any alterations made in an existing structure.

(2) Tests of models can be made which will give satisfactory results when the purposes of the tests are: (*a*) To check an analysis by actual measurement of the behavior of the model under known load conditions; (*b*) to provide data to be used as a basis for changes in design rules; (*c*) to check the efficacy of any proposed alterations in a given construction; (*d*) to supplement theoretical analysis by experimental analysis; (*e*) to avoid theoretical analysis by resorting to experimental analysis; (*f*) to provide an easier, quicker, and usually less expensive means of obtaining the desired information, than would be the case if the actual structure were tested; and (*g*) to provide a means for studying the behavior of a design under loading conditions not possible with a full-sized structure.

In the case of tests of models the results mentioned in Conclusion (2) may be anticipated in most instances, provided the actual departures from strict similarity conditions are suitably taken into consideration when interpreting test observations

Acknowledgments

The writer desires to acknowledge the assistance given in the preparation of this paper, by R. G. Sturm, and E. C. Hartmann, Associate Members, Am. Soc. C. E., and by Mr. M. B. Moore, who made the measurements on the over-sized models of filleted corners.

---

[28] "Factors of Stress Concentration", by M. M. Frocht. *Journal of Applied Mechanics*, A. S. M. E., Vol. 2, June, 1935.

[29] "Vibration of Over-Head Transmission Lines", by R. A. Monroe and R. L. Templin, Members, Am. Soc. C. E., *Proceedings*, A. I. E. E., 1932.

PHOTO-ELASTIC DETERMINATION OF STRESS

By J. H. A. Brahtz,[30] Esq.

Synopsis

The purpose of this paper is to present an up-to-date picture of photo-elastic analysis, to discuss its applications and limitations, to show the relationship that it bears to other methods of stress investigations, and particularly to bring out the relationship that it should, and must, bear to the mechanical and structural designer and his problems. This is a timely subject, since the ever-growing acceptance of indeterminate structures, and the more expensive alloys makes it imperative to the designer that he avail himself of every possible means of refined stress analysis. Since this paper deals with the subject of photo-elasticity, there is naturally considerable space devoted to standard methods, model materials, and technique.

Brief History and Nature of Photo-Elastic Experimentation

Photo-elasticity had its inception more than a century ago, when Sir David Brewster, in 1816, demonstrated that plane-polarized light, on traversing a flat stressed transparent specimen, underwent some peculiar change in its optical properties, such that when viewed through another polarizing unit, colored bands appeared. These bands—isochromatic fringes—he assumed to be a measure of the internal strains. It is now known that these fringes are directly related to the state of stress in the model. The fringe order is, in fact, proportional to the maximum shears throughout the model without regard to their directions, which is equivalent to stating that these fringes (identified as Order 0, 1, 2, 3, etc.), are directly proportional to the difference of the principal stresses.

Just as engineers in the past have generally been slow in applying new mathematical tools, it was not until the beginning of the Twentieth Century that practical application was made of this physical phenomenon. An excellent treatise on this subject, covering a large number of recurrent practical problems, has been written by Messrs. E. G. Coker and L. N. G. Filon[31] who, following closely the pioneer work of Mr. S. P. Thompson[32], have advanced the art and science of photo-elasticity probably more than any other experimenters, and have co-ordinated their results with parallel mathematical analyses based on the mathematical theory of elasticity.

---

[30] Director, Photo-Elastic Laboratory, U. S. Bureau of Reclamation, Denver, Colo.

[31] "A Treatise on Photo-Elasticity", Univ. Press, Cambridge, England, 1931.

[32] "Note on the Application of Polarized Light to Determine the Condition of a Body Under Stress", by S. P. Thompson and E. G. Coker, British Assoc. Repts., 1909.

Theoretical Quantitative Limitations and Practical Qualitative Extensions of the Method

Undoubtedly there are a large number of engineers who are only slightly acquainted with the subject of photo-elasticity, some of whom have very little faith in it as a useful tool. Probably there are others, on the other hand, who believe that it is a scientific achievement representing the last word in stress analysis, and that any problem, regardless of its nature and difficulty, can be put through this seemingly magic process and the answer will appear for them in black and white on a photographic plate. These concepts represent the extremes, and a little clarification of some of its limitations might be in order so as to establish a clear idea of just what the method can do and what it cannot do.

First, the photo-elastic method is a two-dimensional analysis, and the scope of problems must come within the classification of either plane stress or plane strain. For example, a buttress in a dam of the Ambursen type, or the multiple arch, represents a case of plane stress, whereas an imaginary 1-ft slice taken from the center of a long gravity dam is a case of plane strain.

Being an experimental method, coming from the laboratory, the reliability of photo-elasticity analysis is affected by experimental errors and the personal element of the experimenter, the degree to which the operator simulates model conditions to those in the prototype, and the care with which he manipulates his apparatus, will govern the probable error of his results. It is not always a simple matter to simulate conditions directly, or to keep the probable error as low as, say, 10 per cent. For instance, the maximum fringe order may be only four. The experimenter cannot read the fringe order in a photograph closer than one-fourth of a fringe and he may be in error as much as one-half a fringe, which is equivalent to an error of 12.5 per cent. Naturally, this error will be reduced to a minimum when the model is constructed of a highly sensitive material and is loaded close to its elastic limit. Nevertheless, to obtain proper similarity of model to prototype, the physical properties of the model material must be considered carefully. For example, if certain types of experiment are being conducted to simulate a steel structure, the model material should have an elastic modulus that is constant below the yield point. Now, it so happens that the materials that fit this requirement best are not the most sensitive, and those that are extremely sensitive do not behave elastically in a manner similar to steel; therefore, a compromise must be made in choosing a model material that strikes a mean in satisfying the requirements.

Another difficulty is the uncertainty as to just what forces, reactions, and effects are acting on a structure. The same difficulties arise in a mathematical analysis, in which case unknowns are assumed. It would be nice if these factors were always known, but they are not; for example, consider the restraint of the fixed ends of a beam. The fixation is probably never perfect, and no one knows the exact conditions. Numerous examples of this kind can be cited, but they are too well known. However, although these are uncertain conditions, the photo-elastic method can nearly always simulate conditions with more accuracy than any arbitrary assumption.

Corresponding to this indeterminacy of many boundary conditions, there is the difficulty of applying the true loads to models even when they are known exactly. Generally speaking, a concentrated load is much easier to apply than a distributed load, and a uniformly distributed normal load is easier to apply than a uniformly varying normal load or a loading involving shear forces. Furthermore, straight boundaries are easier to load than curved boundaries. These are mechanical difficulties, and the technique is constantly being improved to overcome them. Some new developments along these lines are cited subsequently.

There are some possibilities of expanding the scope of the photo-elastic method in order to deal with certain three-dimensional or composite structures (that is, structures containing materials of varying elastic properties), at least qualitatively. Some progress has already been made along these lines. In this connection mention should be made of some experiments by the late A. H. Beyer, M. Am. Soc. C. E., and Mr. A. G. Solakian, in which they tested simple bakelite beams that contained integral cast-aluminum rods to simulate concrete with steel reinforcement[33].

An experiment has been conducted by Howard G. Smits, Jun. Am. Soc. C. E., in which he attempted to measure the shrinkage stresses in a model of a concrete dam by stretching the base[34]. Although there may be some doubt as to the strict legitimacy of this experiment in simulating shrinkage stresses, nevertheless, as he suggests, it emphasizes the necessity of developing a cement that can be used to combine models of different elastic properties. For instance, in this experiment, the base could be compressed, the dam glued to the base, and then the compression removed. Then tensile stresses would develop in the dam and corresponding compressive stresses in the base. Furthermore, if such a glue were developed, it would open an entire new field of possibilities, such as testing the stresses in a dam resting on a base that is either softer or more rigid than the dam itself; it would also permit the analysis of such problems as a hole in a plate, with a rim of thicker material at the edge of the hole.

Methods, Apparatus, and Model Materials

There is little variation in the optical apparatus used by experimenters in obtaining isochromatic pictures. Essentially, the instrument consists of a monochromatic light source, a nicol polarizing prism, a large lens or concave reflector to create a field of parallel rays in which to place the model, another lens or reflector to focus the rays on another nicol prism, or analyzer, and a camera with a ground-glass back. The reflector type has the advantage of requiring a comparatively smaller space; it produces a large field of parallel rays at relatively low cost, and allows the experimenter to view the ground glass and operate the loading mechanism simultaneously. It is of interest to note that the reflectors are coated with aluminum, in place of the less durable silver, deposited by the method developed by Dr. John Strong, of the

[33] "Photo-Elastic Analysis of Stresses in Composite Materials", *Transactions*, Am. Soc. C. E., Vol. 99 (1934), pp. 1196, 1207.

[34] "Photo-Elastic Determination of Shrinkage Stresses", *Transactions*, Am. Soc. C. E. Vol. 101 (1936), p. 927.

California Institute of Technology, at Pasadena, Calif., in connection with the 200-in. telescope[35].

In addition to giving the value of the difference of the principal stresses or maximum shear, at all points in a model, the polariscope (as it is commonly called) also readily adapts itself to giving the directions of principal stresses at all points, the observer utilizing the black isoclinic lines or lines of constant direction of principal stresses[35]. In many structures the only stresses that are required are those at the boundaries, more commonly referred to by engineers as the extreme fiber stresses. These stresses can be obtained directly from an isochromatic picture, if at a boundary the principal stress required is parallel to the surface, and the other is perpendicular to it; these boundary stresses are equal to zero if the structure is not loaded, or equal to the intensity of normal pressure when loaded normally. Since the normal stress and the magnitude of the difference given by the isochromatic picture are known, the extreme fiber stress is obtained in those cases either by adding or by subtracting the normal load from the measured difference of principal stresses. When the boundaries are loaded with inclined forces, the isoclinic lines must be used to evaluate the boundary stresses, but this is not difficult.

In order to use the photo-elastic method to determine the complete state of stress at interior points an auxiliary step is required in order to obtain the individual principal stresses. Since the difference of the principal stresses has already been found by means of the isochromatics, they could readily be separated by applying a little algebra at any point if one could, somehow, determine the sum of the principal stresses. This auxiliary step can be accomplished by several experimental methods. It is known that in a stressed model, the sum of the principal stresses is proportional to the change in thickness. This change is of an extremely small magnitude, but it can be measured. Coker has developed a very sensitive mechanical extensometer for this purpose, and Professor R. W. Vose, of the Massachusetts Institute of Technology, at Cambridge, Mass., has been successful in constructing a similar extensometer which measures the change in thickness by means of a small auxiliary interferometer[36], which is, of course, the most powerful method of measuring small dimensions. An interesting academic point in this connection is revealed by some experiments conducted by Professor M. M. Frocht, of the Carnegie Institute of Technology, at Pittsburgh, Pa., in which he constructed a test model from a plane-parallel optical flat, and used the two faces of the test pieces for an interferometer. In this manner he obtained interference fringes directly in the model, which again are a measure of the sum of the principal stresses. These bands he calls isopags or isopachic lines[37]. Experiments of this type require a knowledge of Young's elastic modulus and Poisson's ratio of the model material.

---

[35] "Photo-Elastic Apparatus at the California Institute of Technology", by J. H. A. Brahtz, *The Review of Scientific Instruments*, Vol. 5, No. 2, February, 1934; see, also, *Transactions*, Am. Soc. C. E., Vol. 101 (1936), pp. 1268-1277.

[36] "An Application of the Interferometer Strain-Gage in Photo-Elasticity", *Journal of Applied Mechanics*, A. S. M. E., September, 1935, pp. A-99, A-102.

[37] "On the Application of Interference Fringes to Stress Analysis", *Journal*, Franklin Inst., July, 1933, pp. 73–89.

A very practical method of obtaining the sum of the principal stresses is by means of the stretched rubber membrane (see Fig. 16)[33]. A horizontal rigid frame is constructed to the shape of the model and the vertical ordinates of the frame are proportional to the sum of the principal stresses of the boun-

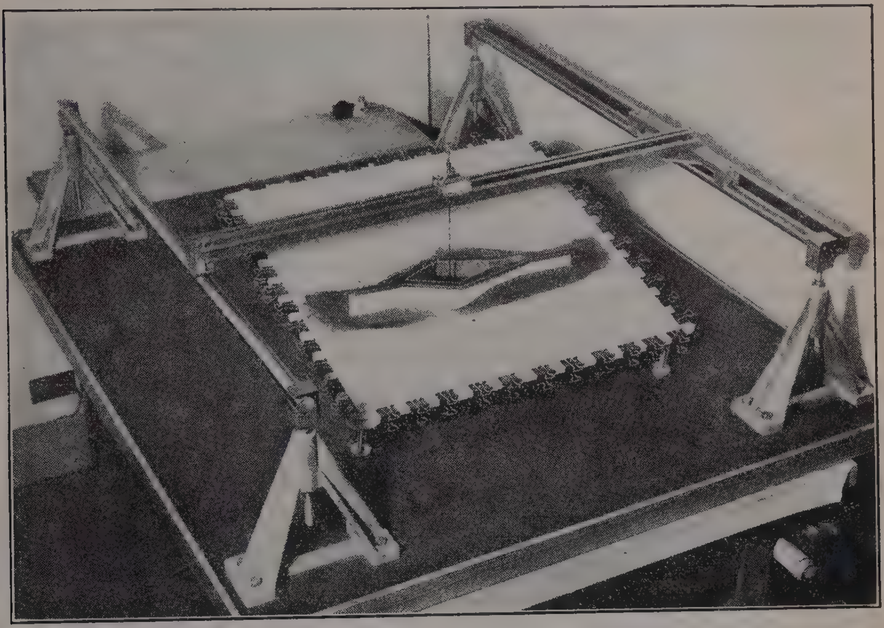


FIG. 16.—MEMBRANE PROFILOMETER (NOTE NEON GLOW LIGHT AND SWITCH IN BACKGROUND.)

daries, which are determined from the isochromatic photographs. A thin rubber sheet is then stretched uniformly over the frame forming a warped surface, every ordinate of which is proportional to the sum of the principal stresses, and these can be measured with an ordinary depth gage.

Although all these methods will accomplish the desired results in completing the photo-elastic analysis at interior points, it is obvious that they have their respective advantages and disadvantages. The two extensometers require a quantitative knowledge of the elastic constants of the model material and depend on very accurate manipulation to reduce experimental errors. Furthermore, only one point on the model can be examined for a given position of the extensometer and each reading requires loading and unloading of the model; with a complicated loading mechanism this involves considerable labor. The only real objection to obtaining the isopachic lines is the cost of constructing the model from a relatively large plane-parallel optical flat. Perhaps the fastest method is the rubber membrane or soap film, but it fails wherever the slopes of the membrane become excessive, and the method

[33] Description of membrane analogy, by J. P. Den Hartog, *Zeitschrift Ange. Math. und Mech.*, Vol. II, 1931, p. 156, and *Journal*, Franklin Inst., July, 1933, p. 5.

depends on very accurate determination of the fringe orders at the boundaries of the isochromatic photographs, which is often very difficult due to edge effects.

Having briefly reviewed the standard methods of obtaining stresses in a loaded transparent model, it is interesting, next, to consider the various model materials in vogue, and their relative advantages and disadvantages. In general, any elastic, transparent, and isotropic material can be used. Of course, this excludes crystalline substances, such as mica, for example. The more common materials are, in the order of increasing optical sensitivity: Glass, celluloid, phenolite, white bakelite, yellow bakelite, marblette, gelatine, etc. Of course, bakelite is the material most commonly used in the United States. Marblette is much more sensitive—that is, it yields more fringes and this reduces the probable error in reading stresses. However, the more sensitive a material is the more it borders upon being a semi-liquid substance and partly takes on the properties of a liquid, particularly surface tensions. For this reason, extremely sensitive materials age rapidly in the sense that the edges tend to round off making it impossible to read the fringe order on the very boundaries. Some of these materials are easier to machine and polish into awkward model shapes than others. Marblette and gelatine, for instance, can be cast. Glass, which is frequently used for obtaining the isoclinic lines that give the directions of principal stresses, is the most difficult to machine and has the added disadvantage of cracking or flying into small pieces while the experimenter is applying the loads.

While on the subject of methods and model materials, a brief digression into body forces might be in order, covering the problem of determining the stresses induced in an elastic structure by its own weight or inertia forces. Naturally these forces cannot be used throughout the model by direct application of exterior forces, but must be accomplished by some other means. None of the materials mentioned previously, with the exception of gelatine, is sensitive enough to produce isochromatics from their own weight with the size and thickness of models ordinarily used. However, gravity forces can be magnified if the model is set in a centrifuge and rotated at a high angular velocity. Some interesting studies using this method have been made by Messrs. Philip B. Bucky, A. G. Solakian, and L. S. Baldin, of Columbia University[39], and independently by Fred L. Plummer, Assoc. M. Am. Soc. C. E.[40]. A more satisfactory method has been suggested by Dr. M. A. Biot[41], in which he applies certain force distributions to the boundaries of the model, obtains the corresponding stresses, and then superposes these stresses to another system of theoretical stresses obtained from a simple gravitational stress function. The net result is the determination of stresses in the model due to gravity alone. This method has been used at the Laboratory of the United States Bureau of Reclamation, in Denver, Colo., and applied to

[39] "Centrifugal Method of Model Testing", *Civil Engineering*, May, 1935, p. 287.

[40] *Transactions*, Am. Soc. C. E., Vol. 101 (1936), pp. 1281–1282.

[41] "Distributed Gravity and Temperature Loading in Two-Dimensional Elasticity, Replaced by Boundary Pressures and Dislocations", *Journal of Applied Mechanics*, A. S. M. E., June, 1935.

the gravity stresses in the Grand Coulee Dam. The results have been very gratifying and there is no doubt that this method is more practical than the centrifugal method.

There is another possibility of accomplishing this result by using thick gelatine models. A step in this direction has been made by R. R. Philippe, of the United States War Department, and Professor Plummer in connection with stresses in the foundations of earth dams[42].

From Model to Prototype

Without entering into too much detail, a brief explanation is included herein on the method of converting observed fringes into stresses in the structure. First, it is necessary to determine what the stresses are in the model. This is done with a simple test specimen cut from the same piece of stock as that from which the model is made. It is customary to make this test piece in the form of a simple beam subjected to pure bending, photograph it, and count the fringe order at the extreme fibers[43]. This same beam is analyzed mathematically and the stress computed at the same point. The stress is divided by the corresponding fringe order, resulting in the stress (the difference of principal stresses) in, say, pounds per square inch in the model per fringe. The two experiments of course, must be made under identical optical circumstances. Now, the law of simulitude may be stated, as follows: The stresses in the model at all points will be to the stresses in the prototype at corresponding points as the normal boundary pressure on the model is to the normal boundary pressure at a corresponding point in the prototype.

Consider a simple example: A beam loaded uniformly with a pressure of 50 lb per sq in. on the edge. A bakelite model is constructed to any scale and loaded with, say, 5 lb per sq in., to obtain the isochromatic pattern. The material is calibrated and, perhaps, it is found that each fringe corresponds to a stress of 200 lb per sq in. in the model. Then, wherever there is a fringe order, 1, in the model, there will be a difference of principal stresses in the structure of 200 × 10 (the load factor), or 2 kips per sq in. If, for example, the fringe order at the center of the beam at the bottom fiber is Order 5, then the stress is 10 kips per sq in. in the prototype.

Comparison with Other Methods of Stress Measurements

It will be valuable to draw some comparisons between the photo-elastic method of stress analysis and two other experimental methods, namely, direct strain measurements on two-dimensional and three-dimensional models and the slab analogy. No mention will be made of the standard practice of testing models to failure, since this is in a class by itself, yielding no stresses, and is based on internal stresses beyond the yield point. In comparing the strain-measurement method with the photo-elastic method, several points should be mentioned. First, the strain-measurement method readily adapts

[42] "Soil Mechanics Applied to Design of Dams", *Civil Engineering*, January, 1936, p. 25; see, also, p. 9.

[43] See, for example, *Transactions*, Am. Soc. C. E., Vol. 101 (1936), p. 1269, Fig. 9.

itself also to three-dimensional models even with varying elastic properties. Because of its adaptability into three dimensions, the method stands alone; it dominates this field completely, and is the only known experimental method suitable for analyzing such a structure as a thick arch dam. Necessarily, it is a tedious expensive method both in execution and evaluation and calls for great skill. It allows elastic relationships between model and prototype to be simulated very closely because of the wide latitude of available materials. Quite often the applied loads produce strains that are scarcely detectable in certain regions and in every case the stresses obtained can represent only average stresses in the segment between gage points. Photo-elasticity yields the stress at a point.

Another method of obtaining experimentally the stresses in a two-dimensional structure is by the "slab analogy." This method is mell adapted to obtaining the stress distribution, for example, in a gravity dam. A flat elastic slab made of rubber, for instance, is cut to the dimensions of the structure and warped by displacing and twisting the boundaries. The curvatures and twists can then be measured at any point within the boundaries of the model, and it can be shown that these curvatures are always proportional to the normal stresses in the structure in a direction perpendicular to that at which the curvature was measured, and that the shear stresses are proportional to the twists in the slab. Naturally, the slab must be set up in such a manner that the twists and curvatures at the boundaries are equivalent to the forces acting on the boundaries. The basis of this experiment lies in the fact that the differential equation of a warped elastic slab has the same form as the differential equation of compatibility of the Airy stress function. This is another one of those analogies that are being adopted by the Engineering Profession, and is in the class with the soap film analogy used in measuring torsional resistance of odd shapes, such as propeller blades and built-up sections. Other well known members of this family are the electric analogy and the aforementioned rubber membrane analogy also used to measure temperatures in the steady state, pressures, and equi-potential lines in highly viscous hydraulic flow that recurs in soil problems[44].

It may be said of the slab analogy that it is an excellent and reliable method because of the directness of the results; but, like the strain-gage method, it is costly and tedious, requiring a well developed technique. In its present state of development it is justified only in analyzing an expensive major structure.

Returning now to the photo-elastic method and without attempting in any way to depreciate the importance and value of the other methods, it can be said of photo-elasticity that it is an inexpensive method of analyzing difficult structures or elements of machines, discounting, of course, the high initial investment in optical equipment. Furthermore, it is a rapid method, especially when one is interested only in extreme fiber stresses. Moreover, it supplies, immediately, a rough picture of stress distribution in a structure and reveals concentrations wherever fringes are grouped close together. To

[44] "The Flow Net and the Electric Analogy", by E. W. Lane, M. Am. Soc. C. E., F. B. Campbell, Jun. Am. Soc. C. E., and W. H. Price, *Civil Engineering*, October, 1934, p. 510.

emphasize these advantages further: Since the investigator immediately obtains a picture of the stress distribution in the model of a structure that is subject, say, to a number of different loading systems, he can, very rapidly, single out the critical cases. Furthermore, the shape of the model can be altered, reduced, or enlarged at weak points that are revealed in the photographs, and the process repeated. In other words, it furnishes a rapidly converging method of design, even when dealing with a structure that borders in the class which, off-hand, one might say is analyzed readily by the Begg's deformeter method, or the Cross method of moment distribution and column analogy. Naturally, the mathematical analysis should be made wherever possible, but now by photo-elasticity a good starting design and the critical loading are quite well isolated.

Practical Relation to Structural Problems

The true value that photo-elasticity must bear to structural and machine design is readily shown to those unfamiliar with the method by a few examples. It will be apparent that photo-elasticity has its own field, and completely dominates this field from a practical standpoint. Conversely, it is foolish to attempt to use it in those fields that are readily treated mathematically, such as standard beams, columns, tension members, etc., of lengths at least five or six times their lateral dimensions, except for academic purpose or as a check analysis. This includes all standard structural members. Where these ratios are shortened, where a structure is composed of short, thick, integral members, where it is otherwise complicated by holes, notches, reinforcements, etc.—in short, wherever it is obvious from the shape of the structure that it is indeterminate in the sense that the stress distribution is non-linear in character, and the principal stress trajectories assume a complicated curvilinear form—then the structural designer finds that his ordinary tools will not suffice and he must depend either upon his structural sense and previous experience, or perform a test to failure, or carry out a photo-elastic analysis. This type of problem is encountered continually in machine design. This does not mean that the photo-elastic method cannot, or should not, be extended to include fields susceptible to mathematical analysis for check or verification. As a matter of fact, the two overlap and, as has already been indicated, it is a rapid tool for finding critical loads and design sections for use in mathematical analyses. Furthermore, a check between the two goes a long way in removing that feeling of uncertainty which can hang like a nightmare over a designer who entertains a feeling, and rightly so, that perhaps he has made a few too many assumptions in dealing with a statically indeterminate problem without knowing whether or not they are on the side of safety.

Consider, for example, the subject of fillets, such as occur in elements of structures and machines. No one doubts their value, and yet an exact mathematical analysis would be extremely difficult. By testing a small bakelite model, however, a picture of the stress concentrations in the fillet can be obtained. It is possible also to see how they compare with stresses in the web, and the operator can begin with a large radius, cut back, and quickly

try any number. Photo-elastic investigations have shown that a spiral fillet in some cases is more efficient than a circular one[45].

As a second example, consider the problem of the chain link of the type with two holes machined from a flat plate (see Fig. 17 and Fig. 18). A photo-

FIG. 17.—ISOCHROMATIC VIEW OF BAKELITE CHAIN LINK.

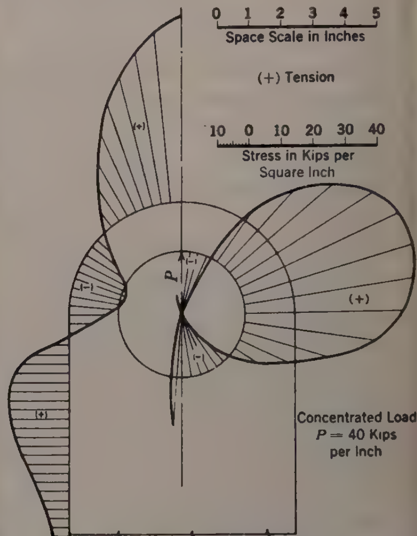


FIG. 18.—BOUNDARY STRESSES IN CHAIN LINK DERIVED FROM FIG. 17.

elastic analysis can readily be used to compare a wide variety of conditions, such as various pin clearances, different ratios of hole to width of link, and various shapes of links. In this type of problem only the boundary stresses are important, and the designer can soon choose an economical and safe shape.

Fig. 19 illustrates an example in which the photo-elastic and mathematical fields overlap, and this experiment was conducted to find critical loading conditions and to check the mathematical analyses made for the critical cases. In this experiment two new loading devices were used[46]. The water pressure in the tunnels was simulated by air pressure applied through rubber inner tubes, restrained on the sides but bearing directly on the walls of the tunnels. The earth pressures were applied through a thick rubber sheet that fitted snugly over the model, and the loads were applied to the outside edges of the rubber.

Other interesting applications of the method are apparent if one side of a bakelite test piece is silvered, and if the light is sent through to the silvered face and back along the same path. For example, a piece of bakelite may be

[45] Discussion of Weibel's paper, entitled "Studies in Photo-Elastic Stress Determination", by A. G. Solakian, *Transactions*, A. S. M. E., August, 1934, pp. 652–655.

[46] "Photo-Elastic Analysis of Twin Concrete Conduit, Bull Lake Dam Outlet Works", by J. E. Soehrens, R. T. Cass, and J. E. Sower, Juniors, Am. Soc. C. E., *Civil Engineering*, September, 1936, p. 594.

cemented on the surface of an actual structure, either two-dimensional or three-dimensional, and the changes observed in the induced isochromatics as the structure undergoes changes in loading; or the investigator may wish to

FIG. 19.—LOADING APPARATUS UTILIZED IN PHOTO-ELASTIC ANALYSIS OF DOUBLE-BARRELED CONDUIT FOR BULL LAKE DAM. (NOTE THAT PNEUMATIC PRESSURE IS USED TO SIMULATE INTERNAL HYDROSTATIC PRESSURE.)

measure the stresses in overlapping plates, such as occur in riveted connections. By silvering he can isolate the stress conditions in the two plates. Furthermore, he can study the behavior of viscous fluids under flow, by photographing the isochromatics that develop as ordinary salad oil is subjected to flow around obstacles set between parallel glass plates. Experiments of this type have been conducted successfully by Dr. Sadron, of the California Institute of Technology.

THE INTERFEROMETER

Before concluding, a word should be said about the interferometer as a means of measuring individual principal stresses in a transparent model directly, thus eliminating the necessity of the auxiliary step of finding the sum of the principal stresses. It has been known for some time that this method can be applied, but in so far as the writer is aware it has never been utilized in the United States. One of these instruments has been designed and built at the Laboratory of the Reclamation Bureau, by J. E. Soehrens, Jun. Am. Soc. C. E. (see Fig. 20). Essentially, the method consists of splitting a pencil of light, one part of which traverses a loaded model at a point, vibrating in the plane of one of the principal stresses, and the other goes around the model. The two pencils are brought together again and interfer-

ence fringes develop. The stress at the point in the model will be proportional to the relative displacement of these fringes as the load is applied. Next, the ray is made to vibrate in the plane of the other principal stress

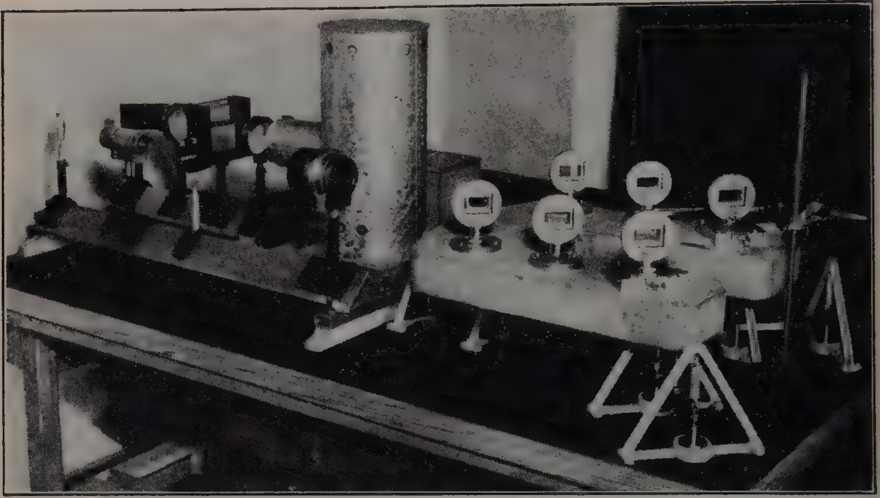


FIG. 20.—PHOTO-ELASTIC INTERFEROMETER.

and the measurements are repeated. It is believed that this apparatus will provide a more accurate method of determining stresses within a model; that is, stresses other than those at the boundaries particularly in cases in which the accuracy of a rubber membrane breaks down, as, for example, at a sharp corner or near the point of application of a load. This machine is much more delicate in its operation than the polariscope and must be designed in such a way that it is insensible to vibrations.

FUTURE POSSIBILITIES

At present, photo-elasticity has demonstrated its value and firmly established itself as an invaluable tool to the Engineering Profession. Its development has been somewhat erratic but positive, and has by no means reached its limit. New model materials and technique are being developed and will continue to be developed. The method is not a simple mechanical one, but a procedure that requires trained technicians. Some universities are now (1936) offering introductory courses in the subject to their undergraduates. As lighter, stronger, but more expensive, alloys assume their natural functions in construction, it will be more important than heretofore to have a complete knowledge of stresses. As these demands become more acute emphasis will be more and more toward photo-elasticity as a means of designing economical structures.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
PAPERS

STRUCTURAL APPLICATION OF STEEL AND LIGHT-WEIGHT ALLOYS

A SYMPOSIUM

PAGE

II.—METALLURGICAL AND MANUFACTURING ASPECTS OF FERROUS AND LIGHT-WEIGHT ALLOYS OF INTEREST IN STRUCTURAL DESIGN AND FABRICATION:

Low-Alloy Structural Steels.
By E. C. Bain, Esq., and F. T. Llewellyn, M. Am. Soc. C. E. 1184

Stainless, High-Alloy Structural Steels.
By M. J. R. Morris, Esq. 1201

Light-Weight Structural Alloys.
By Zay Jeffries, Esq., C. F. Nagel, Jr., Esq., and R. T. Wood, Esq. 1211

Corrosion in Relation to Engineering Structures.
By James Aston, Esq. 1239

LOW-ALLOY STRUCTURAL STEELS

By E. C. Bain,[47] Esq., and F. T. Llewellyn,[48] M. Am. Soc. C. E.

Synopsis

In view of the fact that this Symposium is sponsored by a civil engineering society, the word, structural, in the title of this paper is to be construed s pointing with strong emphasis to framed static structures, such as bridges and buildings, almost to the exclusion of small moving structures and machinery. The other papers in Section II enumerate adequately the more restricted opportunities for over-all economy which are offered to the special high-strength steels in this restricted field of application. Information has been presented to show how the economical expenditure of the steel dollar may still call for the purchase of special steels that differ from common structural steel by having enhanced mechanical properties secured through the incorporation of alloying elements.

Introduction

In this inquiry, the writers begin with the assumption that the background of their subject has already been well drawn; that every one is aware that alloy steels may have a higher elastic range and a higher breaking load than the ordinary structural steels, but that the elastic modulus is the same. The consequences of these circumstances, as they affect design, are presented elsewhere in the Symposium, where the applications of those low-priced alloy steels are discussed. In this paper, the writers discuss the common alloying elements with respect to the direct and indirect influence their presence, both individually and concurrently, may have upon the properties of structural steels.

The writers first approached their task with a plan for a categorical enumeration or classification of the many so-called "grades" of low-alloy, structural steel that are now (1936) on the market, with their compositions and ordinary mechanical properties, as set forth by their makers. Since, however, such information has been given in a number of excellent compilations[49], the writers conclude that it may be more useful to treat the principles involved in designing alloy steels. Accordingly, two types of these steels have been chosen to illustrate these principles, and, as far as possible, the subject has been developed as a functional study. For convenience, however, a table of some of the current compositions offered to the trade and their reported mechanical properties is included.

---

[47] Asst. to Vice-Pres., U. S. Steel Corporation, New York, N. Y.

[48] Research Engr., U. S. Steel Corporation, New York, N. Y.

[49] See, for example, *Proceedings*, Am. Soc. C. E., March, 1936, p. 361.

Although it will be convenient for subsequent reference to compare alloy compositions with the regular structural steel, designated "carbon steel", it may be well to forestall a false premise at the start. The vast majority of the structures within the scope of the present consideration are built of a steel which is by no means the pure metal iron or an alloy of iron and carbon alone; structural carbon steels carry about 0.5% of manganese, one of the important alloying elements. An inspection of hundreds of analyses indicates a probable most frequent composition about as shown in Table 2. Steel of such composition, as rolled, for example, in sections 0.50 in. to 0.75 in. thick, develops mechanical properties such as correspond, on the average, to approximately the tensile test data indicated in this table.

TABLE 2.—COMPARISON OF ORDINARY STRUCTURAL CARBON STEEL WITH PURE IRON

Description (1)	Ordinary, structural carbon steel (2)	Pure iron* (3)
Composition, in Percentages:		
Carbon	0.22	....
Manganese	0.45	....
Sulfur	0.035	....
Phosphorus	0.018	....
Silicon	0.05 or 0.20	....
Ultimate strength, in kips per square inch	60	39
Yield point stress, in kips per square inch	38	19
Percentage elongation in 2 in	45	48
Percentage elongation in 8 in	28	30
Percentage reduction in area	58	75

* Commercial purity.

It is interesting to compare the properties of the usual structural carbon steel with those of nearly pure iron (that is, iron of commercial purity) as indicated by Column (3) of Table 2. It is clear that the ubiquity of carbon in the metallurgy of iron and the availability and use of manganese, primarily as a specific or antidote for possible sulfur ills, have resulted in a considerable gain in desirable properties. One needs no marshaling of metallographic theory to convince himself of the general efficacy of these elements incorporated in steel, but it may be worth while, nevertheless, to digress further into this field of physical metallurgy.

Before having a "look" into the effects of the common alloying elements, it may be advisable to view the problem of stronger steels from the purely economical standpoint, but still through the eyes of the metallurgical engineer.

ECONOMIC FUNDAMENTALS OF ALLOYS

Members of the Society need scarcely be reminded that economy is one of the essentials of good engineering. In considering the economic fundamentals of alloys, or indeed of any materials, it is desirable to emphasize three facts. They need only be stated to be self-evident:

(1) The question as to the economy of a designated material becomes critical only when there is a choice of materials.

(2) If, as a result of its improved properties, the net cost of a substituted material is less than that of the material replaced, there is no economic problem to be solved. Manifestly, despite its higher cost per ton, the material in question would be used everywhere as far as it is available.

(3) A given material may be economical under certain conditions, and uneconomical under others.

It follows that the desirability of alloying steel with certain chemical elements, calculated to produce higher effective strength than ordinary grades, will vary according to the problem in question.

Having in mind these facts, some engineers have surmised that a simple chart could be prepared in which the higher cost per pound of each alloying ingredient, as a commodity, could be translated into final cost per ton per unit of resultant alloying content, and plotted against the corresponding improvement in given physical properties. A chart of this kind would be just what the civil engineer is seeking, but, unfortunately, those who operate steel mills, as well as Dame Nature, say that the matter is not quite as simple as that. Even if effects could be charted against percentage of alloying elements, comparative costs would not be indicated for the reason that rolling-mill costs vary greatly according to the quantities of a kind to be produced at a given time.

Quite apart from mill-operating conditions, however, such a chart is not feasible. As demonstrated subsequently, even the effect of individual alloying elements is involved, to say the least, and is not constant under all conditions. In many cases it depends on the inter-relation of other elements. For this reason, instead of one chart, one would require about as many charts as there are different types of steel, and these charts would be useless as aids in forecasting the properties of a new combination.

Most important of all, it must be emphasized that the improvement in certain properties which is afforded by many of the alloying elements is accompanied by a loss in other desirable properties. If Dame Nature and the economic cartographer attempt to join in holy wedlock, with metallurgy as sole officiating priest, the union cannot be blessed because the progeny would be as numerous and unmanageable as that of the "old woman who lived in a shoe."

Interpretation of Specimen Tensile Tests

It is well to reflect that the ordinary tensile tests, accompanied perhaps by compression tests, supply the engineer with some numbers which are not measures, exactly, of the precise properties which will be depended upon in the final structure, but are instead merely rough indicators of the possession of the needed properties. By way of example, a notch in a beam becomes a stress concentrator under conditions of bending, and failure may occur in a manner developing less than the expected load. On the other hand, in a carefully conducted tensile test, a bar in tension having a sharp circumferential notch may carry, without permanent deformation, a load in excess of the ultimate stress as ordinarily measured. A plate may show 30% or 40% elongation in the standard tensile test; and yet, when subjected to simultaneous

tensional stresses at right angles, it may break with only a slight percentage of elongation as measured along any reasonable reference distance Nevertheless, the standard tensile test, interpreted in terms of experience, is a valuable guide and the writers would defend its general use.

The fact is that the gross loading as ultimately reflected in any small region of the metal may be considered as a force tending to rupture the metal, with a maximum in one certain direction, and a force tending to cause flow (that is, shear) likewise at a maximum, in another direction in a shearing plane. If the maximum shear component exceeds the elastic limit in shear and the metal has any capacity whatever for plastic flow, some deformation, without rupture, will occur. On the other hand, if the maximum disruptive component exceeds the fundamental tenacity or cohesion of the metal, a brittle break will occur. The former may occur first, followed by the latter, by virtue of a change either in the relative magnitude of the two components, or in the elastic limit in shear of the material during the deformation.

In an ideal tensile test in which a uni-axial tension is applied uniformly across the cross-section of a moderately long specimen, the numerical magnitude of the resulting disruptive stress is just twice that of the resulting shear stress. If, therefore, the elastic limit in shear of the specimen is more than one-half that of its cohesion, the tensile specimen will fail with a brittle rupture accompanied by no elongation; and yet, just such a material may possess considerable fundamental plasticity, as can well be shown by applying a torsion test, in which the resulting maximum disruptive stress is numerically equal to the maximum shear stress, on any small element in the highly stressed periphery of the specimen. Other tests, such as bending a notched bar, afford still less opportunity for any flow than does the tensile test, and one must conclude that the quality of realizable plasticity in a broad sense depends actually upon the ratio of the elastic limit in shear to the fundamental resistance to disruptive stress, for the second of which no known means of evaluation exists. This reasoning is introduced only to show that the tensile test supplies only a glimpse from one viewpoint of the mechanical properties of a steel. However, experience has demonstrated that good structures result from material with certain tensile test characteristics and may not result from others, and having no better practical test it is utilized with assurance as far as static loads are concerned.

With respect to structures as a whole, it is rare indeed that any general permanent deformation is tolerable, hence the interest really centers in the stress that can be borne within the elastic range of loading. For this reason increasing reference is made to the so-called "yield point" of the steel. Since the departure of the metal from strictly elastic behavior begins with almost undetectable gradations it is now customary to specify a very small but observable permanent elongation as a working definition of the elastic limit or yield strength; 0.1%, or 0.2% is usual and occasionally a proof stress corresponding to a permanent elongation of 0.01% is specified. The breaking load computed as a stress over the original section of the test has certain interest, to be sure, but of necessity plays a less important rôle in design than the yield strength.

The need for a degree of plasticity will not be detailed in this paper. The inescapable necessity for joints and the proper distribution of stress, otherwise highly localized and concentrated in and about such joints, can only be cared for by some capacity for flow. It is generally believed that 25% elongation in a test piece, 8 in. long, with certain cross-section limitations, is adequate. It should be added, however, that elongation should always be referred to a specimen which has a definite shape and ratio of length to cross-section; otherwise, comparative figures may be quite misleading. The reduction of cross-sectional area at the point of rupture is a factor of some complex significance not wholly clear even to the most erudite metallurgists and not of vital moment to the engineer.

It is not uncommon to measure the energy required to break a small, standard, notched bar by impact and to take this number of foot-pounds (or kilogram-meters) into account as a putative quality index of the steel. Considering that the most plastic and ductile metals known can be made to break in a brittle manner (low impact value) by introducing a sufficiently sharp and effective notch, and considering further the assiduous avoidance of notches in design, it might appear that such notched-bar impact tests would not yield broadly pertinent information. However, the steels under present consideration do not break in the brittle manner under the notches of customary sharpness at ordinary temperatures. If the break at very low temperatures should be brittle, and, therefore, should show low energy absorption (foot-pounds), the engineer should not reject the steel summarily, but instead, he should perhaps break a few specimens, at the same sub-zero temperature, which have been prepared with a more gentle notch and see that it is perhaps the notch, rather than the metal, which has been tested.

It happens that structures do not always bear a constant static load, but are subject to frequent loading and unloading, or even to reversal of load. Under such conditions a failure may occur, after a very large number of load-change cycles, at a specific stress lower than the yield point. Tests of the resistance to repeated stress effects (fatigue) have been devised, and the familiar "endurance limit" expresses this property usually as a maximum fully reversed stress which can be borne in a specimen during 10 000 000 cycles. For many materials this value is found to be well above 45% of the ultimate strength. However, when the stress changes do not entail reversal, but rather result from loadings and partial unloadings (surge loads), the endurance limit applicable approaches the yield point. It has been found that the character of the surface of the stressed member exerts a tremendous influence upon the practical fatigue resistance, and to make the test represent, as nearly as possible, the metal itself rather than the surface condition, a polished test specimen is used. Accordingly. it would seem that for the civil engineer the yield point would be a more significant figure than the endurance limit, but that the endurance limit should be known and taken into account for the material under contemplation if stresses nearly or wholly reversed are to be encountered. Of probably greater significance is the possession of adequate plasticity, such as is measured, in part at least, by elongation and reduction of area in the tensile test.

In an exploratory manner, formability and other criteria of suitability may be judged by the data from conventional specimen tests, but it is to be expected that the final judgment will be based upon tests simulating operating conditions in practice.

The steels discussed in this paper were designed to retain adequate ductility and to provide an increment of yield strength. In practice, they have already achieved a remarkable degree of success. The design of machines (particularly automobiles) long ago opened up a need for steels that could first be endowed throughout large sections with the maximum properties previously securable by careful heat treatment upon only small sections of the higher carbon steels. Actually, it was soon evident that properties somewhat superior even to the best obtainable in small sections of carbon steel were securable in the alloy steels. This was comparatively easy because, by special heat treatment, tremendous improvements over the "as-rolled" properties are securable through the control of the microscopic structure; medium and high-carbon alloy steels with attendant high strength can be rendered tough and plastic as well.

The problem in the field of Civil Engineering is vastly more difficult. Not only is cost a serious deterrent from special heat treatment of structural steel, but the shapes themselves are scarcely amenable to the rapid cooling and reheating involved in such a method of enhancing properties. Since, in most cases, the steel perforce must be used in the as-rolled condition, and since the modern method of joining metals by the newer welding processes must be facilitated, the limitations are very strict. Not long ago it was a commonplace to hear in the metallurgical schools that alloying elements were of practically no value except where heat treatment was to be applied. The statement, never quite justifiable, is now known to be farther than ever from the truth, and to have accepted such a statement would have been to have missed the entire development under discussion in this paper.

Means of Strengthening Steel

For the purposes of this survey, there are only three means of gaining strength in steel: (1) Cold working the members; (2) specially heat treating the members; and (3) incorporating alloying elements.

Cold Working.—Bridge wire is an outstanding example of this application, and the high tensile strength of cables is known to every one. Recently, cold-rolled strip, particularly a certain stainless steel, characteristically amenable to improvement by cold-work, and to spot-welding also, is becoming a very valuable material in movable structures, such as high-speed trains. One can scarcely visualize the extensive cold drafting of standard structural shapes.

Heat Treatment.—To the metallurgist, who realizes that the microscopic structure, and, hence, the properties of steel, depends upon the cooling schedule from an elevated temperature, the cooling of structural steel from the rolling temperature is, in itself, a heat treatment. Although this is proper, heat treatment, in current usage, generally means a subsequent controlled heating and cooling. Thus, for years (since 1914), heat-treated, carbon-steel, eye-bars

have been utilized in bridges, and more than 17 000 of them are now (1936) in place. A 0.35% carbon, 0.60% manganese steel is used, having properties as rolled corresponding to a yield point of about 40 kips per sq in. and an ultimate strength of about 70 kips per sq in.; the ductility of the untreated steel is approximately represented by an elongation, in an 8-in. by 0.5-in. specimen, of about 23 per cent. After the closely controlled heat treatment of the eye-bar, properties are developed corresponding to an average yield point of 57 kips per sq in. and an ultimate strength of 88 kips per sq in. A full-sized tensile test bar with a cross-section, 12 in. by 2 in., and 18 ft long, exhibits a ductility so great that, in the full 18-ft test length, an elongation of nearly 2 ft occurs (10.5%), or in the 1 ft encompassing the point of rupture, 30%; the reduction of area is 46 per cent. A higher carbon steel containing 0.60% carbon, with 0.55% to 0.75% of manganese, is similarly heat-treated to an ultimate strength of about 119 kips per sq in. and a yield point of 82 kips per sq in. The elongation in 18 ft is about 8%, with a reduction of area of 23 per cent.

With such uniform regular sections as eye-bars, heat treatment has proved entirely practicable and economical, but for the usual shapes it is probably not "in the picture" at present; nor is it likely to become so for large welded structures, since any improvement derived from heat treatment would be nullified in and near the weld.

Use of Alloying Elements.—Turning next to the third and most readily applicable means of securing a general purpose steel, with a moderate increase in load-carrying capacity, it is to be noted that the metallurgist has at his disposal the elements—carbon, manganese, silicon, copper, phosphorus, chromium, nickel, vanadium and molybdenum—as well as some other less common elements, as possible addition agents in a structural steel. As far as small increments of equal weight are concerned, the elements are arranged about in the order of their cost, beginning with the cheapest. One should not be led to believe, however, that the order of merit is the same; nor, indeed, is there any basis available for any order of merit whatever unless one single property is under consideration at a time, and unless the proportions of the other elements are held entirely constant for the comparison. If one were to begin with commercial steel of the lowest carbon content and highest purity, and incorporate therein increasing quantities of the various elements, it is apparent at once that the Brinell hardness is increased in the rolled product about as shown in Fig. 21. These curves are drawn as the best probable average of scattered data and should not be regarded as being precise. It is intended to refer to wrought material, hot-rolled to a reasonable thickness, say, $\frac{1}{2}$ to $\frac{3}{4}$ in., and cooled at a normal rate in air. The line for carbon in Fig. 21 refers to smaller specimens so as to reflect its effect upon hardenability. If larger sections are considered, the value at 1% carbon would be about 220 Brinell hardness. Hardness is not correlated perfectly either with yield point or with ultimate strength, but, in general, it increases with alloying elements in a manner such as to correlate somewhat with a value midway between the yield and the ultimate. Fig. 21

merely shows that the fundamental influence of alloys in iron is unique for each element; but such curves fall far short of telling the entire story. Added elements contrive to confer properties by several means, usually operating

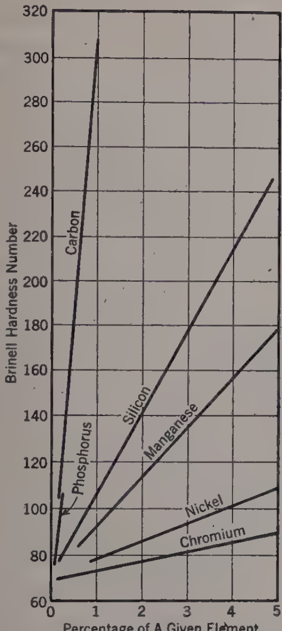


FIG. 21.—APPROXIMATE EFFECT OF SMALL ADDITIONS OF VARIOUS ELEMENTS TO NEARLY PURE IRON.

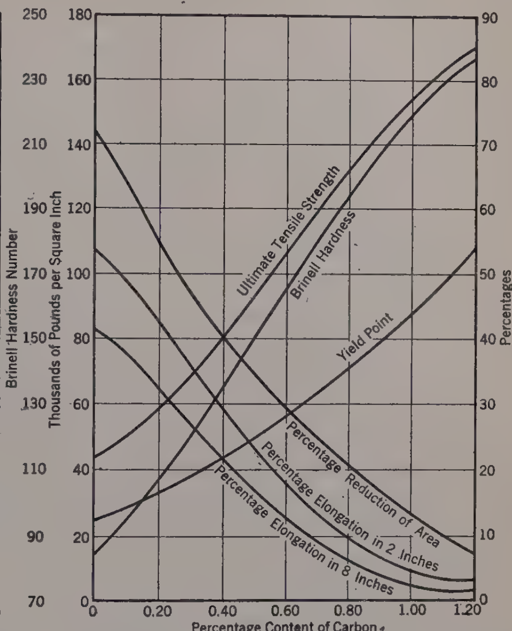


FIG. 22.—APPROXIMATE EFFECT OF CARBON CONTENT UPON THE TENSILE PROPERTIES OF STEEL, ASSUMING NORMAL MANGANESE CONTENT.

concurrently. The principal influences are, as follows: (*a*) Alteration of transformation characteristics (under-cooling and attendant fineness of microscopic structure); that is, "hardenability"; (*b*) solid solution hardness (and strength) increment in the iron itself; and (*c*) modification of the nature and the volume of the carbide constituent of the steel.

Another influence that almost warrants a separate listing has to do with the grain size in the heated condition; a fine grain generally connotes greater toughness.

Influence (*b*) is essentially the one illustrated in Fig. 21, and is imporcase of the higher carbon, higher alloy steels, such as some of the automotive steels—is of first importance. It is an influence which the element exerts by its presence in the steel at rolling temperature by determining the microstructure which finally develops upon cooling through the so-called critical temperature range at a given rate. It would operate to control the properties even if the alloying element were found to have vanished immediately after the steel had been transformed. Rapid cooling always produces finer distribution of carbide and, therefore, harder characteristics for any steel, and any of the alloying elements dissolved in the steel at rolling temperature acts

in a way practically equivalent to a more rapid cooling, because these elements retard the reactions at the transformation, or "critical", temperature. Thus, it is customary to reduce the carbon or the alloy content for the thinner sections which automatically cool more rapidly. This influence must be mild in the steels under consideration and the writers will show that the elements contributing the most to hardenability must be kept low.

Influence (*b*) is essentially the one illustrated in Fig. 21, and is important in the low-alloy, low-carbon steels under discussion. Any element dissolved in a metal raises its strength somewhat (some more than others), and, generally speaking, reduces its ductility although not in a systematic or a perfectly general manner. Some elements reduce ductility more for a given strength increment than others.

Influence (*c*) is also one affecting the strength of the alloy, but its importance increases, obviously, with the carbon element. If an element has a strong tendency to form a carbide then some of it is not dissolved and its contribution by the second mode is thereby decreased by increased carbon. If, however, in replacing the normal iron-carbide lamellæ with a special carbide, the form or volume of such carbide is also changed, and then some practical property changes are possible. Obviously, carbon itself cannot be considered in precisely the same way as is possible for the other elements because it is the necessary constituent of all carbide in the steel. In general, strength in steel gained by increasing the quantity of carbide is accompanied by a greater decrement in ductility than is the added strength from dissolved elements. The metallurgist will recognize that the carbide in the steels as rolled from the compositions discussed herein will exist in the form of the minutest microscopic platelets or lamellæ, a form not nearly so conducive to ductility as the spheroidized form found in fully heat-treated steels. In the spheroidized condition, as may be surmised, the carbide is in the form of roughly spherical particles barely visible under the microscope. It will be understood that these features of microstructure are nothing that can be seen by the eye in the fractured surface, but are known only through application of the metallurgist's technique of examining a polished and etched section microscopically. Perhaps a philosophy of alloy steels can be developed by considering the elements, one at a time.

Characteristics of Individual Alloying Elements

Carbon.—Fig. 22 shows the approximate change in tensile-test properties created by the carbon content in steels of normal manganese (0.45%) and silicon content as rolled in sections of the order of 0.75 in. thick. As might be predicted from Fig. 21, carbon is the most powerful element, weight for weight, utilized in influencing the properties of steel; it is customary, however, to exclude carbon when referring to alloys or alloying elements. Its marked effect upon strength properties is gained at a considerable toll in ductility, and long experience indicates that at about 0.20% to 0.25% (with comparatively low content of other elements), a broadly applicable optimum is reached for structural carbon steels. At this content, a considerable range

of other elements and of section is still permissible, and it happens also that the influence of carbon upon hardenability is not so great at this point as to preclude welding, bearing in mind the low content of other elements. Perhaps one may state that carbon contributes practically nothing by the second influence, that of solid solution, since at ordinary temperatures it is so nearly insoluble, and that the quantity of carbide as a microscopic constituent is proportional to the carbon content and, hence, a strong effect results in this (the third) manner. Furthermore, even with a minor content of other elements, carbon exerts a definite influence toward hardenability; only in its presence, and in proportion to its concentration, are other elements able to exert substantial influences toward hardenability.

Nevertheless, when need has arisen, engineers have not hesitated to use steels of higher carbon content. Because of the added requirement of wear resistance, rails carry about 0.75% of carbon; the eye-bars, already mentioned, are high in carbon since heat treatment eliminates a large part of the loss in ductility brought about by carbon. In cases wherein no welding is to be used and wherein joining problems are not complicated, carbon content higher than the usual can always be used with safety. However, where welding is to be utilized, the usual structural steel derives about all the strength that carbon alone can provide without excessively restricting other valuable properties.

Manganese.—After carbon and phosphorus, manganese is one of the most effective strengtheners of steel, rivaling chromium. Its effect may be greater than shown in Fig. 23 (*a*), which is an attempt on the part of the writers

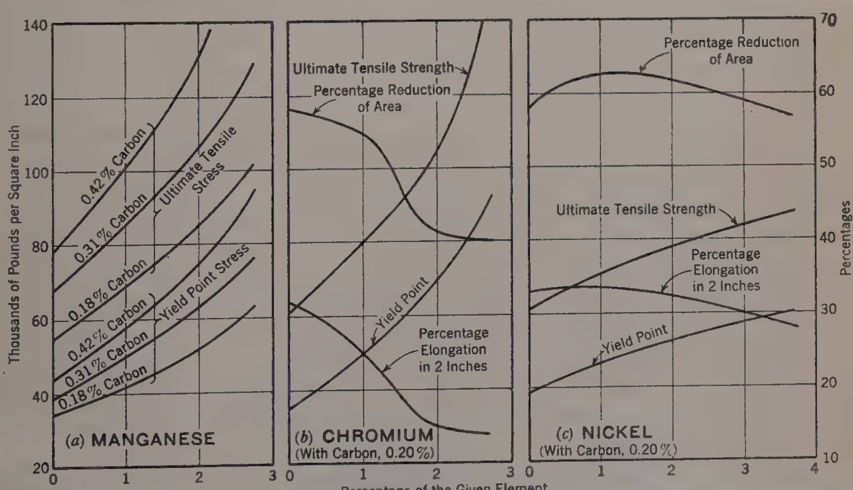


FIG. 23.—APPROXIMATE EFFECTS OF THREE DIFFERENT ALLOY ELEMENTS UPON THE TENSILE PROPERTIES OF CARBON STEEL. NORMAL MANGANESE IN CHROMIUM AND NICKEL CHARTS.

to correlate some scattered information. Manganese is an extraordinarily powerful hardening agent (Influence (*a*)), and it also strengthens greatly by solid solution. It has no particular carbide effect, although it divides itself between the iron solid solution and the carbide. It has the disadvantage, how-

ever, of taking a considerable toll of ductility in the as-rolled condition, apparently through its solid-solution effect. In general, it should probably not be used much in excess of 1% to 1.3% in any non-heat-treated, general-purpose steel, and for the greatest ductility, perhaps 0.50% to 0.75% is sufficient. In a broad way, the over-all manganese effects are not unlike those of carbon; with very low-carbon steel, manganese is perhaps one-eighth as effective as carbon in developing strength, but in somewhat higher carbon steel it is certainly more like one-fourth or one-fifth as effective, weight for weight, because its hardenability effect enters. In order to preserve good ductility in the manganese steels the carbon must be decreased somewhat and more than the equivalent effective manganese is then to be added in order to gain strength; but even this substitution has strict limitations. In the ideal high-strength, general-purpose steel, other helpful elements should be called into use.

Silicon.—This element is found almost entirely in solution in the iron (ferrite), and its solid solution effect is strong. In small amounts (that is, as much as about 1%), it appears to reduce ductility only moderately, and some report that it has a curiously minor effect upon the reduction of area in the tensile test. It has a significant effect upon hardenability which is, however, small in comparison with that of manganese, carbon, or chromium, and in the presence of these elements its effect upon hardenability through its influence upon transformation rate must be taken into consideration. It is a valuable aid in the low-carbon steels in securing strength, and may be utilized safely up to 0.75% or, in some cases, nearly to 1.0%, depending upon the other alloy content. If it has any effect upon corrosion rate, it is not obviously adverse. Silicon has been rather less fully reported than the other elements.

Copper.—This element remains entirely dissolved in the iron when present in quantities as great as, perhaps, 0.50% or 0.60%, in which proportion its contribution to strength is small indeed, perhaps only 2 to 4 kips per sq in. In larger proportions it may be utilized as a precipitation hardening element with very substantial strengthening effects. In the lower range, however, it contributes to a valuable reduction of corrosion, a property again lost when the quantity is well into the range for precipitation hardening. Obviously, the age-hardening feature cannot well be utilized in the structural steels under consideration in view of the undesirability of subsequent heat treatment and the requirements attending welding. One may state, then, that copper may profitably be used up to at least 0.5%, with a valuable gain in corrosion resistance, a slight gain in strength, and no perceptible loss in ductility.

Phosphorus.—The traditional fear of phosphorus brittleness was largely dispelled by the classical work of Unger in 1918[50], who showed that phosphorus in quantities considerably greater than 0.10% contributed strength without undue loss of ductility. Indeed under some conditions an increment in strength could be provided by means of phosphorus with a penalty against ductility less than if the same increment were gained through a higher carbon

[50] "Effect of Phosphorus in Soft Acid and Basic Open-Hearth Steels", by John S. Unger, *Year Book*, Am. Iron and Steel Inst., 1918, p. 172.

content. In other words, a reduction of carbon content with a deliberate addition of phosphorus yields a steel in which strength is increased and ductility remains constant; it is then a suitable element for the structural steel under discussion. More recently the effect of phosphorus to reduce corrosion has been recognized, and since this is a factor of increasing importance, incidental advantage is taken of its quota of strengthening effect, while securing at the same time a marked improvement in corrosion resistance. At about 0.07% the strengthening effect of phosphorus is such as to necessitate a reduction of carbon content, but quantities as great as 0.15%, or even more, are still useful under some conditions.

Chromium.—This metal exerts its greatest influence through its contribution to hardenability, and large additions demand either a low carbon content or a heat treatment. In higher carbon steels, its strong carbide-forming tendency is outstanding, but this effect is of less consequence in steels carrying only 0.15%, or less, of carbon. Its straight, solid-solution effect in iron is extremely mild as may be seen in Fig. 21. This is particularly true in the case of the yield point which is scarcely changed by a small percentage of chromium in very low carbon steels or in extremely slowly cooled higher carbon steels. The ultimate strength, however, is more markedly affected. Fig. 23 (*b*) illustrates the approximate influence of chromium upon the properties of a 0.20% carbon structural steel as rolled to a 0.5 or 0.75-in. section In steels used in the as-rolled condition, it acts to control the microscopic structure and to foster the stronger structure in thin sections. Like manganese, therefore, it may best be used with a reduced carbon content.

The effect of high percentages (as, for example, 11%, or more) of chromium to develop an inert surface is so marked in the corresponding stainless steels that its modest contribution to corrosion resistance in the lower chromium steels passed almost unnoticed until recently. As an auxiliary element to raise strength with little loss in ductility and to encourage a harder structure from the transformation, it is useful and may be employed safely in quantities as great as about 1% in the present class of steels whenever manganese is not already relied upon for most of the strength increment.

Nickel.—This element, like silicon, exerts little or no influence upon the quantity or character of the carbide formed in low carbon steel. It is somewhat more effective as a solid-solution hardener than chromium, but has rather less influence upon hardenability. Its effects are mild but positive, and strength gained by nickel is often accompanied by very little or no loss of ductility. The only disadvantage in its use is the relatively large quantity required to secure marked increase in strength. Its improvement of corrosion resistance is well known and compares with that of copper except that it is not so effective, weight for weight. Where it may be used in relatively large quantity, and where ductility is extremely important, it is particularly well suited. Fig. 23(*c*) shows the approximate effect of nickel upon the properties of a 0.20% carbon structural steel as rolled to 0.5-in. to 0.75-in. section.

Molybdenum.—As a carbide-forming element, molybdenum exhibits a grain-refining influence probably next to vanadium in intensity. Even with low carbon steels the molybdenum, as ordinarily used, is to be found chiefly in

the carbide constituent where its effect is an indirect one through the grain-size effect mentioned previously. In this way, molybdenum may counteract some of the hardenability effects and promote a more advantageous distribution of the carbide than that of plain carbon steels. When it is dissolved at high temperature, however, as in the hotter zones of a weld, it exercises a hardening influence. In view of the cost of molybdenum, it is customary to use it very sparingly, in the alloys under consideration. The grain-size effect can be achieved, in large part, by other means, such as measured deoxidation utilizing aluminum in a special way. The outstanding value of molybdenum, revealed by its tendency to form persistent carbide globules, is manifest in the creep-resisting steels used at elevated temperature.

Vanadium.—The effect of vanadium, as ordinarily used, is essentially that of grain-size reduction, with its attendant control of hardenability and preservation of toughness. By this indirect influence it permits a more generous use of manganese, but does not enter the picture of low-priced structural steels to any great extent. It becomes a more important element in the higher carbon steels used in the heat-treated condition.

The foregoing comments show that the acquisition of high strength is easily securable by the addition to iron of practically any of the elements enumerated, but that in the case of the most effective ones the strength is secured at a considerable sacrifice in ductility. Carbon accomplishes its results through an increase in the non-ductile constituent carbide, whereas the other elements act more gently through solid solution effects and by a change in hardenability. To secure a greater strength with maximum preservation of ductility in the as-rolled condition, carbon can be used only sparingly, and even manganese and phosphorus must not be used in quantities contributing very greatly to strength.

Complex Steels: Simultaneous Addition of Several Elements

A fortunate circumstance is that the concurrent use of moderate quantities of several elements appears to produce a more favorable combination of properties than would result from a single element in an amount sufficient to produce an equivalent strength increase. Roughly speaking, in the small quantities under discussion herein, the effects of the elements are sufficiently additive to permit rough first approximations of properties from composition, but even those who introduced the early formulas for predicting tensile test data from analyses recognized some inter-relation which rendered linear formulas unreliable.

Suppose, for example, that the objective is a steel of 55-kip yield strength. This is easily accomplished by increasing the carbon content to about 0.50% and holding manganese to 0.45%, but by so doing the elongation in 2 in. is reduced to about 20%, and less for 8 in., a figure regarded as inadequate for general purposes by many engineers; but such a steel will not yield a weld of desired properties in the metal adjacent to the weld because of undue hardenability, and the semi-fused zone may be still more susceptible to the loss of ductility. If one uses about 1.60% to 1.85% of manganese and about

0.20% of carbon, the 55-kip yield point may be achieved and the elongation in 2 in. will not be far below 25%; but, on occasion, the weld may still not be quite as free from undue hardening as might be desired. Further improvement can be secured by further reducing the carbon and adding silicon; or, finally, by using still less manganese and adding yet another element, a very attractive set of properties is securable. Thus, for example, in a chromium, manganese, silicon steel (Cromansil), as rolled in 0.5-in. to 0.75-in. section, with carbon, 0.12%; manganese, 1.0%; chromium 0.75%; and silicon 0.75%, one would expect to secure about such tensile properties as ultimate strength of 80 kips per sq. in.; yield point, 55 kips per sq. in.; elongation, 26% in 8 in., and reduction of area, 55 per cent. Slight improvement may result from a further moderate decrease in manganese and increase in chromium and silicon. Similar improvements are obtained by the use of nickel in rather higher proportions, say, 2% to 3%, and some corrosion resistance is secured simultaneously. Whatever combination is utilized the corrosion resistance seemingly may be improved by the incorporation of as much as 0.50% copper and by the use of 0.10%, or more, phosphorus. A steel with which the writers happen to be familiar, is constituted in accord with the foregoing line of reasoning, with particular attention to high-yield strength and corrosion resistance, and has an average analysis approximately, as follows:

Composition (Percentages):	
Carbon (maximum)	0.10
Manganese	0.30
Phosphorus	0.10
Silicon	0.75
Chromium	1.0
Copper	0.35
Average Properties in Ordinary Sections:	
Yield-point stress, in kips per square inch	55
Ultimate strength, in kips per square inch	72
Percentage elongation in 2 in.	25
Percentage reduction in area	60
Izod impact, in foot-pounds:	
At 70° F.	60
At 40° F.	40
Endurance limit, in kips per square inch	45

There are obviously many alloy combinations from which to choose in securing the mechanical properties set as the objective of the new structural steels. In selecting examples thus far the writers have only expressed their concept of a logical design of alloy-steel composition. It should be borne in mind that there are few, if any, sharp maxima in the ductility-strength combination when properties are regarded as a function of composition. Indeed the optimum composition ranges for the principal properties are probably broad ones, and the minor desiderata may have to be taken into account

to enable a final choice to be made. An impression of the various alloy compositions offered to the trade may be gathered from an inspection of Table 3[51]. The writers are not responsible for certain apparent discrepancies that appear in Table 3.

TABLE 3.—Composition and Properties of Some Commercial Structural Alloy Steels

Chemical Analysis (Percentages)									Mechanical Properties						
Carbon	Manganese	Silicon	Chromium	Copper	Nickel	Molybdenum	Phosphorus	Vanadium	Yield point stress, in kips per square inch	Ultimate tensile stress, in kips per square inch	Percentage Elongation in: 2 inches	Percentage Elongation in: 8 inches	Percentage reduction in area	Impact Test, in Foot-Pounds: Izod	Impact Test, in Foot-Pounds: Charpy
0.08	....	....	....	1.0	2.0	....	....	....	61	75	32	......	60	....	43
0.15	....	....	....	1.0	2.0	....	....	....	64	82	29	......	60	....	41
0.22	....	....	....	1.0	2.0	....	....	....	61	88	27	......	57	....	....
0.12	1.0	....	....	0.5	0.5	0.20	....	....	60	75	25	......	......	75	....
*	*	....	....	1.5	1.0	*	....	....							
0.30	1.0	....	....	0.5	0.5	0.20	....	....	70	90	18	......	50	....	....
*	*	....	....	1.5	1.0	*	....	....							
0.12	0.5	0.30	....	0.9	0.45	....	0.10	....	69	78	......	22	68	42	36
*	0.7	*	....	1.25	0.65	....	0.15	....							
0.12	0.15	†	....	0.3	0.3	0.05	0.05	....	47	67	28	......	......	....	....
*	0.90		....	0.8	0.8	0.25	0.15	....							
0.25	0.75	0.25	0.25	0.3	0.25	....	0.08	....	70	90	20	......	......	50	....
*	*	*	*	0.5	*	....	0.10	....	50	65	25	......	......	50	....
0.35	1.25	0.30	....	0.40	....	....	....	....	50	80	20	......	......	40	....
*	1.75	*	....	‡	....	....	....	....							
0.14	0.7	0.15	0.12	0.25	....	....	....	....	70	80	19	......	......	....	....
	0.9	0.20		0.30	....	....	....	....							
0.20	1.2	0.15	0.12	0.25	....	....	....	....	85	105	12	......	......	....	....
0.30	1.6	0.20		0.30	....	....	....	....							
0.08	0.6	0.50	0.25	0.40	0.25	§	....	....	50	65	20	......	......	....	....
0.30	0.9	*	*	0.60	*	....	....	....	72	90					
0.10	0.10	0.5	0.5	0.3	....	....	0.10	....	50	65	22	......	60	60	....
	0.30	1.0	1.5	0.5	....	....	0.20	....	60	75	27	......	......	40	....
0.35	1.25	0.10	....	0.25	....	0.40	....	0.20	55	90	20	......	......	....	....
*	1.70	0.30	....	*	....	*	....	*							
0.20	0.40	0.20	....	0.25	....	....	....	....	45	90	18	......	......	....	....
0.40	0.80	‡	....	*	....	....	....	....							

* = Maximum. † = Trace, ‡ = Minimum, § = Molybdenum present

The writers are persuaded that the low-carbon content of most of these steels is an important feature and one that is worthy of careful consideration. Not only is high carbon unnecessary in view of its replacement by larger additions of alloying elements in securing high yield strength (say, 55 kips per sq in.), but ductility in the as-rolled condition in thinner gages depends upon its being kept low; but still more important is the manner of hardening during rapid cooling after welding, which often is not easily preventable. Some engineers regard about 0.15%, maximum, of carbon as highly desirable in the general case, although a higher percentage may be satisfactory in some instances. Clearly, the lower the carbon content, with physical properties constant, the less will be the opportunity for dissatisfaction in this respect.

[51] "Low-Alloy, High-Yield Strength Structural Steels, An Extended Abstract", *Metals and Alloys*, March, 1936.

At the same time wherever welding is not used, or in cases amenable to stress-relief annealing following welding, the higher carbon steels are applicable, and they may possess still higher strength.

It is noteworthy that most of the low-carbon structural alloy steels manifest a relatively high endurance limit. Some of them are surprisingly high and others show an endurance limit of approximately 50% of the ultimate strength. Considering that comparatively few structures have to withstand full reversal of load, it is not believed that special consideration need be given this aspect of the question in the substitution of the alloy steels for ordinary structural steel.

Manufacturing Aspects

From the standpoint of the user, the manufacturing operations at the steel mill, involved in the production of the structural ferrous alloys covered by this paper, do not differ greatly from those used in the production of ordinary structural steel. There are differences, of course, in the manner in which the alloying elements are applied, in the furnace or in the ladle, in discard practice, and, in some cases, in the precise temperature at which the rolling process is finished, but it is believed that the civil engineer, who designs, fabricates, or erects structures in which the alloys in question are used, is not primarily concerned with these features. Indeed, operating practice differs in the several producing plants. Those who are interested may refer to the meritorious outline of mill practice in the production of the steels in question, which appeared in the Progress Report of Sub-Committee No. 2, Committee on Steel of the Structural Division, on Structural Alloy and Heat-Treated Steels[52].

Of greater interest to the civil engineer is the extent to which the special grades in various forms are readily obtainable from the steel mills. The fact that manufacturing operations are similar to those required for ordinary grades does not mean that the special grades are as readily obtainable; they are not as easy to manufacture in a wide variety of section for several reasons:

(1) At present, the demand for any given special grade is not as great as for ordinary grades; therefore, they are not rolled as frequently, or in as wide a variety of sections; and,

(2) The execution of orders for small tonnages of a given special grade must either await the accumulation of other similar orders, or the required items must be taken from stock. The probability of the existence of a stock of the desired grade, and in the desired form and length of section, is much less than in the case of ordinary grades.

For these reasons it is believed that prospective users should consult with the respective steel manufacturers as to the availability of a special grade of steel, for a designated structure, before they incorporate members of that grade into their design.

One interesting fact not always realized by users relates to the excess material produced. When an order for a given lot of special steel is taken,

---

[52] *Proceedings*, Am. Soc. C. E., March, 1936, p. 361.

the steel mills regularly produce more than the quantity ordered so as to provide against possible rejections. If no material is rejected the excess is either scrapped or placed in stock. The latter practice explains the occasional offering by the manufacturers of odd lots of a special grade.

Summary

By incorporating moderate proportions of the common alloying elements, already used in other alloy steels, and at the same time reducing the carbon content, structural steels are now manufactured, which possess increased strength, particularly yield strength, with little or no diminution of plasticity or ductility. Weldability and formability are maintained by virtue of this altered composition. Under suitable conditions these gains in desirable properties may make possible an over-all economy in the use of alloy steel instead of the structural carbon steel in spite of its higher cost.

When advantage is taken of the superior properties to reduce the thickness of members greater attention must be paid to corrosion resistance, because corrosive attack does not reduce the effective metal section in proportion to thickness, but irrespective of section. Fortunately, alloying elements can be selected which confer a reduced corrosion rate to the steel, and this is a valuable property wherever a protective paint or other surface cannot be maintained continuously.

A number of alloy combinations are effective in achieving these results; manganese, nickel, chromium, and silicon are probably the most effective strengtheners, as ordinarily used, and copper, phosphorus, molybdenum, and vanadium are useful auxiliary elements. Copper, nickel, phosphorus, and chromium contribute important resistance to corrosion.

In these low-alloy steels, when the carbon content is less than about 0.15%, and when the high yield strength is derived by judicious combination of the other elements, no serious hardenability interferes with welding processes, the formability is more than adequate, and the product is not unduly sensitive to commercial variations in composition or gage. The content of carbon or the powerful hardening elements should be regulated by the needed ductility and the needed freedom from air-hardening characteristics.

Acknowledgments

The writers are indebted to the Union Carbide and Carbon Company for the use of Fig. 23(*b*).

STAINLESS, HIGH-ALLOY STRUCTURAL STEELS

By M. J. R. Morris[53], Esq.,

Synopsis

In the realm of civil engineering the products discussed in this paper are new. The alloys generally used to-day are ferrous products of the mild structural steel type, which have definite physical qualities and stable surface characteristics. The Engineering Profession, therefore, has evolved a design within their limitations and, it may be stated, the curve of progress is now flattening out. Any hope of marked improvement in design involving this grade of steel does not appear possible.

Since about 1915 a class of steels has been produced in which the physical properties of the base (plain carbon) have been changed by the addition of alloys. This field of endeavor has been much capitalized in the structural steels for automotive engineering applications, but, otherwise, little use of these properties has been made by civil engineers; and yet, this evolution in automotive practice accounted for the most important change in steel metallurgy within the last two decades, amounting in 1929 to 1 ton in 15 tons.

At the beginning of the World War a new chapter was opened, wherein the base plain carbon was alloyed with chrome, etc., to produce not only increased physical strength but also increased stability of surface. Unfortunately, this group has been termed stainless steels. They are not a single steel, but a large group, the base of which is approximately 12% chromium. This is the "corner stone" of the group which will be discussed in this paper.

Introduction

Steels designed under the specifications of the Society of Automotive Engineers have definite and desirable physical characteristics, but they do not possess the capacity to withstand various forms of corrosion; and it is this latter set of properties which enables this group to be used in lighter sections because of their better physical characteristics. It can thus be seen that one of the great drawbacks encountered in thinner sections of low alloy, high tensile steels—surface deterioration—is not found among the high alloys, and now for the first time a group of steels is being developed which will have increasing fields of application.

Perhaps more than any other, the aircraft industry has exploited the use of stainless steels and of other high tensile alloys. This industry has been followed recently by the endeavors of the railroad companies. The Eads Bridge, at St. Louis, Mo., was a pioneer in raw products. It is a chromium

[53] Chf. Metallurgical Engr., Central Alloy Div., Republic Steel Corp., Massillon, Ohio.

steel structure. Alloying 12% chromium with base plain carbon produces a new alloy, its ultimate tensile strength being capable of variation from 50 to 160 kips[54] per sq in. by heat treatment. At the same time, this alloy is highly resistant to atmospheric, and ordinary water, corrosion attacks (referred to base plain carbon life, it would be infinite).

This grade is in its simplest form, and can be further modified by increasing the chromium content with, or without, increasing the carbon. These properties are shown in Table 4. The chromium series are modified with small quantities of nickel (say, as much as 2.5%) to improve the physical properties and with molybdenum and silicon to increase the stability of the surface. These alloys are ferritic in structure and magnetic, and generally respond to heat treatment as a means of increasing their physical properties.

TABLE 4.—CHEMICAL AND PHYSICAL QUALITIES OF CHROMIUM AND CHROMIUM-NICKEL ALLOYS

Item No.	Alloy No.[f]	Carbon[a]	Chemical Analysis (Percentages)								Physical Properties	
			Chromium		Nickel		Silicon[b]		Other Elements		Yield Point Stress, in Kips per Square Inch	
		(1)	From: (2)	To: (3)	From: (4)	To: (5)	From: (6)	To: (7)	From: (8)	To: (9)	From: (10)	To: (11)
1[g] ..	12 Cr	0.12	12.0	14.0	....	....	......	......	......	......	40	170
2[g] ..	12 Cr	0.12	12.0	15.0	....	....	(0.20)[j]	(0.40)[j]	0.45[l]	0.65[l]	40	150
3[h] ..	18 Cr	0.12	16.0	18.0	....	....	......	......	......	......	40	100
4[i] ..	28 Cr	0.20	23.0	30.0	....	....	......	......	......	......	....	45
5[h] ..	18-8	0.20	17.0	19.0	7 0	9.5	......	......	......	......	35	225
6[h] ..	18-8	0.20	17.0	19.0	7.0	9.5	......	......	0.15[m]	0.30[m]	45	130
7[i] ..	18-8	0.20	17.0	19.0	7.0	9.5	2.0	3.0	......	......	..	40
8 ...	18-8	0.11	16.0	19.0	9.0	14.0	......	......	2.0[l]	4.0[l]	40	130
9[h] ..	18-8	0.11	17.0	19.0	7.0	10.0	......	......	(4)[n]	(4)[n]	35	200
10[h] ..	18-8	0.15	17.0	19.0	8.0	12.0	......	......	(6)[o]	(10)[o]	40	200
11[i] ..	25-12	0.20	22.0	26.0	11.0	14.0	......	2.0[k]	......	......	....	40
12[i] ..	25-20	0.25	24.0	26.0	19.0	21.0	......	[k]	......	......	....	40

TABLE 4—(*Continued*)

Item No.	Physical Properties (*Continued*)						Creep Strength for 1% Elongation per 10 000 Hours, in Kips per Square Inch, at							Modulus of elasticity, in pounds per square inch
	Tensile Strength, in Kips per Square Inch		Percentage Elongation in 2 Inches		Percentage Reduction in Area		900° C	1 000° C	1 100° C	1 200° C	1 300° C	1 400° C	1 500° C	
	From:[c] (12)	To:[d] (13)	From:[c] (14)	To:[d] (15)	From[c] (16)	To:[d] (17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
1[g]	70	200	28	10	65	25	....	12	5.0	2.2	1.6	1.1	0.7	28 000 000
2[g]	70	175	25	10	60	30	....	....	....	....	....	....	....	28 000 000
3[h]	70	125	28	10	60	25	....	9	5.3	2 0	1.4	1.0	0.6	29 000 000
4[i]	......	75	28	....	60	....	....	....	3.5	1.7	0.8	0.2	....	29 000 000
5[h]	85	275	55	6	65	30	24	18	11.0	7.0	4.5	2.6	0.9	29 000 000
6[h]	100	160	55	20	60	40	....	..	....	....	....	....	....	29 000 000
7[i]	......	90	50	....	60	....	....	19	12.0	7.0	4.6	2.5	....	29 000 000
8	90	160	55	20	65	50	....	....	....	....	....	....	....	29 000 000
9[h]	85	250	55	5	65	30	....	....	14.0	8.0	6.0	3.0	....	29 000 000
10[h]	85	225	55	5	65	30	....	....	....	....	....	....	....	29 000 000
11[i]	......	90	55	....	65	....	....	....	....	....	....	....	....	29 000 000
12[i]	......	90	50	....	55	....	....	21	13.0	7.5	0.5	2.8	....	29 000 000

[54] 1 kip = 1 000 lb.

TABLE 4—(*Continued*)

Item No.	Coefficient of Expansion at A° C		Magnetic properties	Welding properties	Heat treatment properties	Heat conditions,[e] in calories per cubic centimeter per degree Centigrade	Scaling Temperatures, in Degrees Centigrade		Brinell Hardness Number		Izod Impact, in Foot-Pounds	
	Values of A From 0° to:	Coefficient, ×10⁻⁶					Continuous	Intermittent	From:[c]	To:[d]	From:[c]	To:[d]
	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)
1[g]	800	12.6	Yes	Poor[p]	Good	0.05	1 400	1 400	160	418	100	30
2[g]	800	12.6	Yes	Poor[p]	Fair[r]	0.05	1 400	1 400	160	340	70	20
3[h]	800	11.2	Yes	Poor[p]	None	0.045	1 500	1 600	160	255	70	1
4[i]	1 000	13.0	Yes	Poor[p]	None	0.04	2 000	2 100	160	...	30	1
5[h]	800	20.0	No	Good	None	0.035	1 600	1 400	156	418	110	10
6[h]	800	20.0	No	Fair[q]	None	0.035	1 400	1 400	179	350	80	10
7[i]	800	20.0	No	Good	None	0.035	1 700	1 500	156	...	80	..
8	800	20.0	No	Good	None	0.035	.....	.....	160	350	110	50
9[h]	800	20.0	No	Good	None	0.035	1 600	1 400	156	396	100	10
10[h]	800	20.0	No	Good	None	0.035	1 600	1 400	160	350	90	15
11[i]	1 000	20.2	No	Good	None	0.03	2 000	1 600	160	...	90	..
12[i]	1 000	22.5	No	Good	None	0.03	2 050	1 700	160	...	90	..

[a] Maximum allowable. [b] Except Item No. 2. [c] Annealed. [d] Heat-treated or cold-drawn. [e] Soft steel, 0.10 calories per cu cm per degree Centigrade. [f] Trade designation by which the alloy is commonly recognized. [g] Heat-treated to improve physical properties. [h] Cold-worked to improve physical properties. [i] These items will harden on cold-working but since they are used only for heat resistance they are annealed only. [j] Sulfur. [k] Maximum. [l] Molybdenum. [m] Selenium. [n] Titanium four times the carbon content. [o] Columbium six to ten times the carbon content. [p] Brittle welds. [q] May check. [r] Fair to good.

In the austenitic group of stainless steels the base is chromium plus nickel (approximately 18% chromium plus 8% nickel) with less than 0.20% carbon. These steels have low yield points, but high tensile strength, in the annealed condition; but they have an extremely high degree of ductility. They are incapable of gaining increased physical properties by heat treatment; they can be hardened by mechanical work only. The range of physical properties is large as shown in Table 4. This base alloy has been modified by molybdenum, columbium, titanium, silicon, and copper, each of which contributes some desirable modification, depending upon the different field of application.

Molybdenum is added within a range of 2 to 4% to increase its resistance to wet corrosion particularly of the sulfurous acid type. Columbium is added in quantities equal, roughly, to ten times the carbon content in order to affect the stability of the carbon at elevated temperatures. The effect is to prevent separation of carbon at the grain boundaries in the form of carbides, which separation is undesirable, as this condition is more susceptible to corrosion. Titanium is added for the same purpose as columbium and in quantities generally five times the carbon content. It was the first ingredient so used and may be supplanted by columbium. These end points are attained also by lowering the carbon content only, but the drawback is that grain refinement is not maintained at the same time; hence, the foregoing modification was developed.

Silicon is added to increase resistance to dry corrosion or scaling at elevated temperatures. There is no marked increase in strength from the addition of this alloy. Copper is added at the present time for the purpose of increasing resistance to wet corrosion, and possibly definite data on its rôle will be forthcoming in the near future.

Physical Properties

The physical properties of the alloys are given in Table 4. One item that should be emphasized especially is the elastic limit of the mechanically worked austenitic type; the degree of mechanical working necessary to attain this value properly is in excess of 25 per cent. It is then noted that the elastic limit is indefinite and the curve is continuous. The evaluation of this characteristic can readily be seen to be widely divergent, depending upon the method utilized. This fact is shown in Fig. 24(*b*). The characteristics of types of stainless steel that are hardenable by heating are shown in Fig. 24(*a*). This is of the same type as in all the steels patterned after the

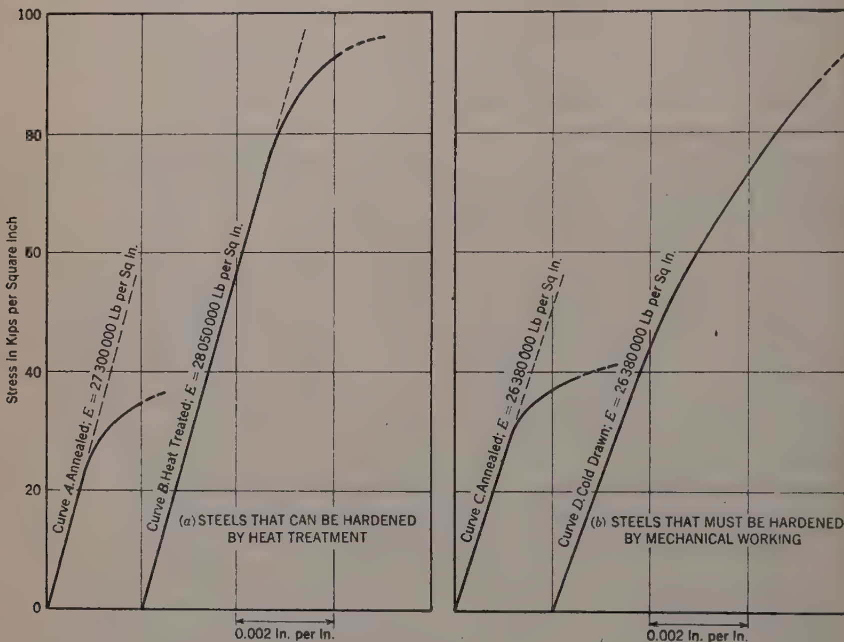


Fig. 24.

specifications of the Automotive Society of America, but, as already mentioned, the austenitic steels can be hardened only by mechanical working. Therefore, referring to the basic plain carbon structure, a margin of superior physical properties is found which is several times the present base.

In addition, these austenitic steels are easily weldable by the modern electrical or gas-welding methods. They can also be formed, punched, and forged without difficulty, but it should not be forgotten that they are stronger steels and, therefore, will require more energy to work. It is safe to state that they are being handled successfully to-day.

It is now logical to discuss briefly the most important property of this group of steels; that is, their corrosion resistance. No one steel in this field has properties adaptable to every case and it is for this reason that the word,

"stainless", has been unfortunate and, in fact, has hindered their proper development and use. It is just as improper to state that there is only one acid. Metallurgists know better to-day, and this important fact being appreciated is leading to more intelligent application and thorough understanding of these steels.

The civil engineer has various atmospheric conditions confronting him—from pure country air to sea air—and highly industrialized conditions. He encounters problems of diverse characteristics, from the practically non-corrosive to extremely corrosive types. These conditions can be analyzed much more readily to-day than in the past. Lastly, he meets a variety of soil conditions, which can be also measured much more intelligently to-day for their corrosion conditions. It can be seen, therefore, that knowing these conditions, the metallurgist and the engineer can design a more suitable steel product to serve a particular purpose.

Since about 1925, much has been done to stress the corrosion fatigue properties of steels. This emphasis has increased, definitely, the importance of obtaining the necessary information as to the medium of application. The surfaces must have a maximum degree of corrosion resistance.

Manufacture of Stainless Steels

Because of their high alloy content, all the stainless steels discussed in this paper are made in an electric furnace of a maximum capacity of 20 tons, generally approximating 10 to 15 tons. The ingots are generally rolled, but the more special types are much stiffer and are forged from the ingots into blooms, which are then rolled.

Being much stiffer than the ordinary plain carbon steels or structural alloy steels, stainless steels are slower to absorb heat and have a narrower working range. The mills, therefore, are different from those used for manufacturing plain carbon steel. They are usually slower and more powerful. It is necessary to have very good temperature control and good fuel conditions. These steels demand a more complete understanding of the mechanical working operations; otherwise, no resemblance of success is obtained and much disappointment results. However, these are days of definite control of operations, and rule-of-thumb and good luck should not, and cannot, exist.

It has already been stated that the rolling requires extra strength, but that the annealing and cooling conditions must have definite control. It is a matter of general practice to handle straight chromium types as all ferritic and hardenable steels are handled—that is, they are cooled very slowly. In the austenitic types, however, the reverse is necessary in the case of nearly all sections in present use. When cooled slowly, the crystalline structure of these types is endangered and rendered more or less useless, but this condition is so easily tested that there is absolutely no excuse for a poor product being shipped from the steel mill or by the fabricator. Generally, it can be stated that sections containing less than $\frac{30}{1000}$ in. cool rapidly enough in air so that this harmful condition is avoided. The thicker sections are quenched to increase their rate of cooling. Hence, it is seen that the two annealing treatments are opposite.

The entire group of these alloys is complex and at the present time their methods of manufacture and production are truly in the evolution period or flux; it is encouraging to be able to state that rapid progress is being made. In a period of little more than 10 yr a product has been transformed from the jewelry classification to one of promising expanding industrial uses, and it is computed that the production for 1936 is well on the way to 100 000 tons.

Application of Stainless Steels in Civil Engineering

The application of stainless steels should be divided into fields of endeavor. It has been shown in recent years that, on certain jobs, definite use can be made of this group of steels, and all such applications have been major investments of the more permanent type. It would be better to divide the various uses into fields that have passed beyond the pioneer stage and describe the better known uses in each field.

Dam Construction.—A proper field for the application of stainless steels has been in the construction of dams, such as in gate-rollers, guides, seal strip, and screw mechanism for raising and lowering the gates. All this use applies to the gate structure and mechanism. It is conceivable that the larger gates could be made of structural steels overlaid with stainless steels and, although there are not yet a sufficient number of cases upon which to base sweeping conclusions, it is felt that such a suggestion would be accorded a high preference. These applications are seriously dependent upon freedom from corrosion in order to keep the mechanism free to operate, galling and rusting being fatal. The rollers and guides usually contain 12% chromium, heat-treated to a Brinell hardness number of about 350. The seal strip is austenitic, with low carbon content, of the type analyzed in Items Nos. 5 to 10, Table 4.

At present, expansion joints between concrete blocks in dam construction are being made of a thin gage strip of low carbon steel. Several of the projects in the Tennessee Valley Authority have this form of construction. It has been thought that, in rock-fill dams, the water face should be of thin stainless sheets, and it is reputed that one such application is being installed in the Severn River, in Great Britain, as an experiment. The ducts controlling the water to and from the locks should present a proper application of stainless steels as liners; this is being considered in some designs at present.

One of the largest uses of stainless steel in dam construction has been on the Aswan Dam, in Egypt, in which about 2 500 tons were used. The dam is 1.25 miles long, and was built in 1892. At the end of 1928, an International Commission inspected it and reported the necessity of further raising and strengthening it. Accordingly, it was decided to erect masonry buttresses between the sluices, but instead of these buttresses being fixed to the face of the dam, they were to be laid against a layer of stainless steel plates placed against the face. The buttress thus was to become a live load. These alloy plates, with a high chromium and nickel content, are 7 mm (0.280 in.) thick, and of varying sizes to suit the dimensions of the masonry buttresses in conjunction with which they are to be used.

More than 100 tons of 4 to 6% of chromium steel were used in the construction of a dam completed in 1935 at Hawk's Nest, W. Va. The steel was used as concrete reinforcing rods and represents one of the first applications of alloy steels to this type of work. The 4 to 6% chromium steel was selected because, by its use, a high tensile strength (180 kips per sq in.) could be obtained without special heat treatment and also because the chromium content renders these rods much more immune to the rusting that always occurs in such applications.

Great quantities of stainless steel were used for sluice-gates, rollers, etc. on the Arapuni Power Project, in New Zealand. This is a unique installation because of the great head of water. The gate is the most heavily loaded in the world, the total water pressure on the face being 2 000 long tons.

In bridge design, the rockers and rocker-plates have recently been made of stainless, heat-treated steel. Some experts maintain that the solid floors of bridges can be modified profitably by the use of stainless grating structures, which are claimed to be stronger, considerably lighter, more rust resistant, and more resistant to wear. One bridge has been so modified. This development received its first important impetus from experiences with the George Washington Bridge over the Hudson River, in New York, N. Y.

Stainless steel wire was used for auxiliary counterweight cables on the Buzzard's Bay Bridge, at Cape Cod, Massachusetts. This steel was the type analyzed in Items Nos. 5 to 10, Table 4, and the cables were $1\frac{5}{8}$ in. in diameter. These cables are in a position not readily accessible for inspection and the use of stainless steel is an added safety factor.

Sewage Disposal Plants.—Stainless steel products should find definite application because of their anti-fouling properties. Heavy pipe lined with stainless steel has been used in large quantities in the City Sewage Plant, at Milwaukee, Wis., because of its low friction factor.

Intakes.—In water systems supplying cities, etc., the austenitic and straight chromium types of steel have been considered. These steels have been applied on intake screens, the type used depending on the design; and generally they have been the austenitic type. The filter screen in artesian well installations has disposed of considerable stainless steel. These screens are of several designs, either wire or sheets being used. Wells of this construction are very easily cleaned. In filter beds, thought has been given to utilizing perforated stainless steel for the floors, the perforation being of such size as to eliminate certain of the coarser gravels.

In 1935, a huge intake crib was completed in Lake Michigan off the foot of Chicago Avenue, Chicago, Ill. It covers the intake from which water for the municipal supply is drawn from the lake through a tunnel below the lake bottom. Older cribs were constructed of granite. A unique feature of this attractive new concrete and steel structure is the stainless steel ring that lines the shaft. It serves to protect the shaft and to resist the rust and erosion that could be expected to attack other forms of construction. It is 39 ft in diameter, 10 ft high, and weighs 3 700 lb.

Reinforced Concrete Structures.—In reinforced concrete structures several problems are presented which may be helped sometimes by the use of stainless alloy steels. When ordinary steels rust, the products of this rusting occupy a decidedly larger volume than that occupied by the non-rusted product from which it came. This produces expansion stresses which may result in cracking the concrete. If the superimposed thickness of the concrete is slight, the steel work is soon exposed; if the concrete covering is thick, this rate of rusting is much less. However, this thought has been embodied in the construction of one dam and, although the alloy was not the true stainless type of steel, its use was a pioneering effort which embodied the principles.

In constructing a new reservoir near Sheffield, England, the engineers decided to demolish an old weir on the River Derwent and to build a measuring channel instead. This channel is unique in that its masonry bed is covered with a stainless steel sheet, the purpose being to prevent the growth of moss and other vegetation that would choke the flow and induce errors in the measurements. This application of stainless steel, although unusual, is entirely practical and is indicative of the extent to which foreign engineers are utilizing its advantages.

Finally, large roofs demonstrate the first use of metals possessed of corrosion-resisting properties. Lead was an early form—then copper, brass, and other non-ferrous products. Necessarily, all this preliminary experience was with such metals as these because of the known attributes of steel and iron products. In these special applications, as ably demonstrated by the roofs of the Pennsylvania Railroad Station, in New York, N. Y., steel and iron rusted quickly and were not easily worked. Stainless steels could be used in such cases with a saving in weight of at least 10 per cent.

St. Paul's Cathedral, in London, England, is a splendid example of how masonry structures that are weakening from age may be strengthened by stainless steel. A few years ago Government inspectors discovered that the huge dome of this famous old cathedral was showing signs of deterioration. Not only was the life of the magnificent structure threatened, but the fractures in the dome constituted a serious danger to worshippers. Tests were made both with tie-rods of strong common alloy steels and with rods of special stainless chrome-nickel steel. The rough oxidizable surface of the common alloy steels adhered to the cement readily, which at first seemed to be an advantage. However, it required a maximum load of only 2.85 tons to withdraw the plain rods. Then bars of stainless steel were rolled and indented so that a series of flat rods was formed. To withdraw these bars from the hardened cement took a maximum pull of 18.88 tons. The result is that stainless steel tie-rods, 40 ft long, threaded at each end, and 4 in. in diameter, are now firmly embedded in the walls of St. Paul's Cathedral to tie the inner wall to the outer wall of the dome.

Irrigation.—Much stainless steel is being utilized in irrigation practice partly to resist corrosion. The Shanan Plant, on the Uhl River, in Punjab, India, is an excellent illustration of this development.

Conclusion

It would appear, therefore, that stainless steels of various composition, permit the selection of a type with attributes that will fill the demands of any given engineering problem. At present it is not as simple to supply them as in the case of the plain carbon steels because the cost would be excessive. When the cost is secondary, or when a high first cost is justified by hazards to life, they can be applied. However, although the application of stainless steels to safeguard life is increasing, this is not the only field. In large structures, weight will be materially reduced by designing these alloys in conjunction with the low carbon types. As in the case of the railroad and the airplane, conventional designs will not be followed but types of structures will be evolved. This will enable a more widespread use of stainless steels. The problem now is to learn to use their valuable properties.

LIGHT-WEIGHT STRUCTURAL ALLOYS

BY ZAY JEFFRIES[55], ESQ., C. F. NAGEL, JR.[56], ESQ.,
AND R. T. WOOD[57], ESQ.

SYNOPSIS

This paper is intended primarily to present those metallurgical principles and properties of the light-weight alloys which govern: (1) The selection of the most suitable composition, temper, or form of material; and (2) the selection of fabricating practices and degree of control required if the best results are to be obtained. As the paper embraces both aluminum and magnesium, and as their characteristics differ so widely, they will be dealt with separately.

PART I—ALUMINUM

HISTORY

Many aspects of the history of aluminum have been amply covered in the literature, especially in 1936, which marks the Fiftieth Anniversary of Hall's noteworthy discovery[58], an event which extracted this metal from the limited enjoyment of the few and made it available as a common article of commerce.

In spite of these previous writings, it is considered worth while to summarize briefly those pertinent events and developments that caused aluminum to become a structural engineering material. It may well be remembered that, for years, the position of this metal in the industrial "sun" was almost limited to essentially non-engineering applications, as, for example, the humble household cooking utensil. During those years, it was not deemed reasonable that such a light metal could do the world's "heavy work." It appears that it was not until aluminum alloys were used in the construction of aircraft that they were conceded the possibility of becoming engineering materials in any field.

In 1898, an Austrian, named Schwartz, constructed a rigid airship with the framework and outer covering of aluminum. Unfortunately, this ship was destroyed on its first landing and, hence, this effort should probably not be recorded as establishing the validity of aluminum alloys for engineering purposes. However, on July 2, 1900, Count von Zeppelin, after many difficulties, launched his first rigid airship. Gaged by the standards of that day the aluminum alloys, from which the framework of this ship was constructed, functioned satisfactorily. The composition of these early aluminum materials varied somewhat, but they were principally aluminum-zinc alloys. They were not heat treatable, but obtained their strength from the effects of the added alloying elements, coupled with cold working.

[55] Associated with Aluminum Co. of America, Pittsburgh, Pa., and Gen. Elec. Co., Schenectady, N. Y.

[56] Chf. Metallurgist, Fabricating Div., Aluminum Co. of America, Pittsburgh, Pa.

[57] Chf. Metallurgist, Magnesium Corp., Pittsburgh, Pa.

[58] "Fifty Years of New Product Development", by Messrs. Frary and Edwards, *Chemical and Metallurgical Engineering*, February, 1936, p. 64.

In 1909, Alfred Wilm, of Germany, brought forth an aluminum alloy, since known as duralumin, which was infinitely superior to those formerly available. The superior combination of qualities of this alloy, such as high strength, workability, and resistance to corrosion, very quickly indicated its usefulness as an aircraft material. This alloy, and the process by which it is manufactured, is of particular historical interest for two reasons: (1) It marks the first noteworthy instance in which the strength of an aluminum alloy was increased materially by means of a heat-treating operation; and (2) in later years, the explanation of this behavior, originated by P. D. Merica[59] and his associates, revealed to the world a new metallurgical principle and one which not only profoundly affected later developments in the aluminum industry but also the metallurgical advance of many other metals.

Marked progress has been made during the past fifteen years in acquiring an understanding of what occurs within the structure of the metal during and subsequent to the heat-treating operation. This has made possible an intelligent selection and control of the alloying ingredients and manufacturing processes, resulting in an ever growing list of improved and more useful products.

Especially during the early years of this development period the principal urge for higher strength, greater uniformity of properties, increased resistance to corrosion, and greater diversity in the form and range of sizes came from the aircraft industry. This was not unnatural. Several conditions in the aircraft industry immediately following the cessation of the World War, which corresponded approximately with the commercial introduction of the heat-treatable aluminum alloys, all conspired to bring about an intensely active development on a co-operative basis between the aircraft and the aluminum industries. The aircraft industry was in its infancy; the application of the metal was of even more recent date. There are few engineering fields in which the demand for perfection of properties is necessarily so highly exacting, in which the materials of construction must be utilized so efficiently, and in which the factor of safety must, of necessity, be calculated to such a fine point. At the same time, the penalty of a failure is terrific.

The industry proceeded with the best materials then available, setting specifications that called for the most constant vigilance for their fulfillment, but pressed insistently for continued improvement and advance. The aluminum industry responded, especially in the United States, and there ensued a development which resulted in the commercial production of a variety of new aluminum products not hitherto available. The guaranteed minimum yield strength was increased more than 100% in less than twenty years. New products were produced, possessing such excellent resistance to corrosion that they could be used bare without any protective coating whatever and this, in sheets, as thin as 0.0095 in.

The object of relating this history is not primarily to praise the aircraft industry (although praise is due), but to indicate the standards of achieve-

[59] "Heat Treatment of Duralumin", *Paper 347*, National Bureau of Standards, November 15, 1919.

ment sought during this development period. These same standards have governed the development of other aluminum alloys for other engineering applications.

The demands of other industries also contributed considerably during this period, and increasingly so. The railroads became actively interested in light alloy construction at least as early as 1923. Rightfully, they emphasized initial, as well as ultimate, cost; they stressed length of useful service far beyond that required for aircraft during that period. Aviation's first problem was to make something fly; and, next, to fly well and safely. Now it has reached the stage of adding "and economically."

Other industries came in for increased attention: The truck, the bus, the marine structure, construction equipment, building materials, sewage disposal equipment, and now the bridge industry.

Each field presented different problems, problems that differed as to technical characteristics of the material that were of governing importance, as to requirements dictated by design and feasible fabrication practices, and as to competitive costs. The result has been the development of a relatively large list of alloys and tempers, available in a very extended range of forms and sizes, some especially suited to one field, some particularly adapted to other fields. With the production of these new materials has come a substantial fund of knowledge of the metallurgical principles involved, an understanding of which is very helpful in the intelligent selection and use of these materials.

Important Metallurgical Principles of Structural Aluminum Alloys

Effect of Composition.—The more common metals used for alloying with aluminum are copper, silicon, manganese, magnesium, zinc, nickel, and chromium. Although occasionally only one of these alloys is added, frequently two or more are introduced, the quantity of each depending on the results desired. It is not possible to predict, definitely, the exact properties of a new composition, nor just how it will respond to various manufacturing processes; and yet certain general principles have been developed. The addition of any one of these elements results in increasing the ultimate strength, yield strength, hardness, shear, and fatigue or endurance values, usually at the expense of elongation, reduction of area, plasticity, and malleability. Magnesium is one exception. Although the strength is regularly increased (see Table 5), in conformity with the general principle, by the addition of

TABLE 5.—Effect of Addition of Magnesium Upon Mechanical Properties of Aluminum-Magnesium Alloys in the Annealed Temper

Magnesium content (percentages)	Tensile strength, in kips per square inch	Yield strength, in kips per square inch	Percentage elongation in 2 inches	Magnesium content (percentages)	Tensile strength, in kips per square inch	Yield strength, in kips per square inch	Percentage elongation in 2 inches
0.75	15.00	6.00	30.0	5.30	38.70	17.50	27.0
2.50	26.76	10.70	25.7	6.00	42.00	18.50	29.0
3.00	28.90	11.65	26.7	8.00	46.00	21.00	32.0
4.25	35.50	15.25	26.0	10.00	52.00	24.00	32.0

more magnesium, the elongation at first decreases, but later recovers. The modulus of elasticity is only slightly affected by alloying. The greater the quantity of the alloying element, the greater in general will be the effect upon the several properties. The magnitude of the effects also varies with the particular alloying metal; that is, the addition of, say, 2% of copper will not affect the mechanical properties to the same degree as the addition of the same percentage of magnesium. Each alloying agent possesses its own quantitative influence.

The aluminum most resistant to corrosion is that of highest purity; in other words, the general effect of alloying is to lower this resistance. Again, the several metals vary in their effects. The alloys with copper and zinc are among the least resistant; the manganese and the magnesium alloys are on a par with commercially pure aluminum. The alloy consisting of 1.25% manganese (3S)[60], possesses a resistance to corrosion quite equal to that of commercially pure aluminum (2S). Likewise, the alloy (52S) consisting of 2.5% magnesium plus 0.25% chromium, and the alloy (53S) that consists of 1.25% magnesium, 0.7% silicon, and 0.25% chromium, and can be heat-treated, all possess a resistance to corrosion that places them in the same class as commercially pure aluminum. Corrosion resistance is one of the properties of these alloys that frequently dictate their selection.

Although the corrosion-resisting qualities of any particular new alloy cannot be definitely predicted in advance, many individuals and organizations throughout the world have concentrated on studying the specific behavior of present commercial and experimental compositions when subjected to a wide variety of laboratory and service conditions, with the result that there is available a wealth of useful information from which the prospective user can determine fairly well whether a particular alloy will meet his requirement satisfactorily.

Another effect of change in composition that may be of importance is that upon weldability. All the commercial aluminum alloys can be welded by the various methods—such as torch, electric spot, seam, butt, and arc—but the facility with which this can be accomplished and the properties of the resulting joint vary considerably, depending on the composition and, in some cases, on the welding method. For example, some magnesium-bearing alloys are less readily weldable by the torch than most other aluminum alloys, whereas they lend themselves especially well to joining by the electric spot and seam methods.

Welding means joining by fusion, regardless of the source of heat. Hence, during the act of welding, a portion of the parent metal becomes molten and, later, re-solidifies. Some compositions are rather tender or "hot short" at temperatures close to their melting points and cannot easily withstand the relatively high local contraction stresses set up during solidification and cooling. This is the case with some of the copper and magnesium-bearing alloys, whereas the silicon-bearing alloys behave differently. An alloy (43S) of 5%

[60] The symbols in parentheses denote the trade designation by which the alloy is commonly recognized.

silicon possesses a relatively small solidification contraction, a low melting point, and a wide melting range.

Therefore, when a particular job is rather difficult to torch weld, because of the characteristics of the alloy or the complexity of the assembly, recourse is frequently had to a 5% silicon alloy (43S) for the welding wire. This is often the choice when the parts to be assembled must be held rigidly in a jig, a situation that prevents free movement of the parts to compensate for expansion upon heating and contraction upon cooling.

In any welded joint, there is a portion of metal that is in the form of a casting, unless the joint is of such a design as to permit the hammering, and thus the conversion, of the metal into its wrought state. Therefore, consideration must be given as to whether the properties of the particular alloy in the form of a casting will be satisfactory. Some compositions (99% aluminum) are as ductile and tough in the form of a casting as in their wrought state, whereas others have a relatively low elongation. Furthermore, the heat-treatable alloys, when in the form of a casting, do not respond to the heat-treating operation as quickly, nor to the same degree, as when in the wrought state. In the case of a torch weld, fusion takes place completely

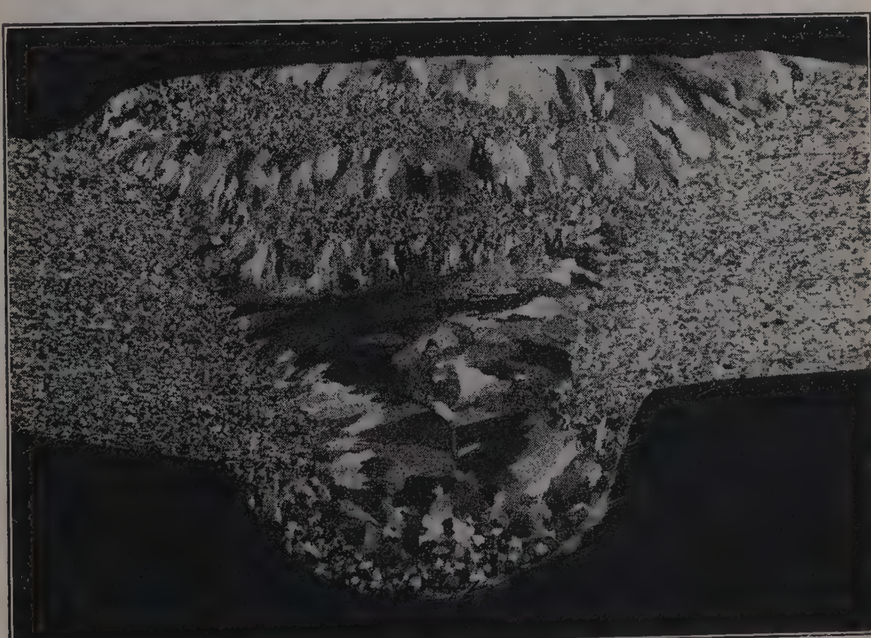


FIG. 25.—CROSS-SECTION OF BUTT TORCH WELD IN A 99% ALUMINUM SHEET. WELDING WIRE IS ALSO 99% ALUMINUM. (MAGNIFICATION, 7.5 TIMES.)

through the thickness of the parts being joined. On the other hand, electric spot and seam welding cause the melting of only a very small portion, and this area, in a properly made spot, does not proceed to the surface of the joint. This is illustrated in Fig. 25 and Fig. 26, which show in cross

section a torch and a spot weld. The details regarding these matters have been well developed, and are available in current literature[61]. In Fig. 25, note that the metal in the welded portion has been melted completely through the thickness. In the case of the spot weld the area affected became molten

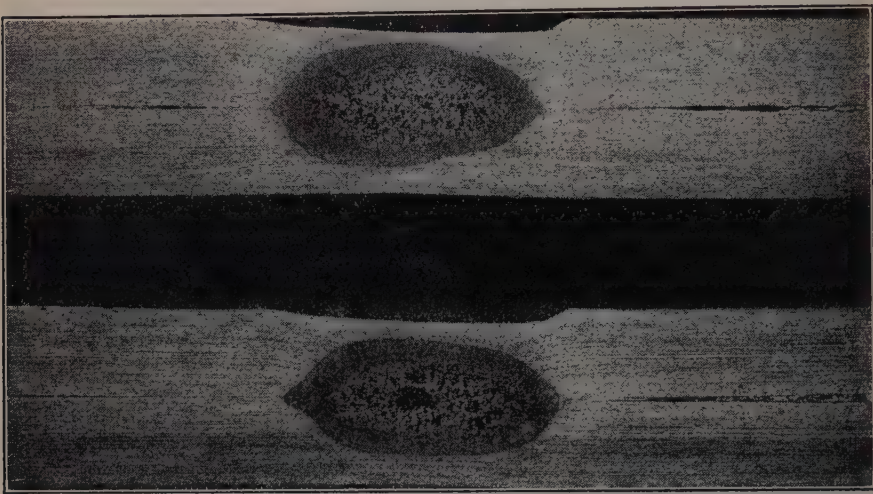


FIG. 26.—CROSS-SECTION THROUGH A LAP, ELECTRIC SPOT, WELD IN AN ALUMINUM-MANGANESE-CHRONIUM SHEET. (MAGNIFICATION 10 TIMES.)

during the process. In Fig. 26, note that this dark area has not proceeded to the surface of the joint.

One of the major justifications for using aluminum in structural applications lies in its relatively low specific gravity in comparison with the other commercial metals. Although the specific gravity of the various aluminum alloys differs, this difference is so slight as seldom to constitute a factor in deciding whether to utilize one aluminum alloy rather than another. The greatest difference in this value among the present commercial aluminum alloys is about 3.5 per cent. Some of the alloys in which magnesium or silicon are the alloying agents are actually lighter than pure aluminum (see Table 6).

The coefficient of thermal expansion is also affected by the composition but here, again, the difference is so small as not to influence one's choice in favor of any particular alloy (except, perhaps, in some very special case), nor to argue against the selection of several different aluminum alloys in the same structure.

Other physical properties affected by composition are the electrical and thermal conductivities. These effects may be substantial. Typical values are shown in Columns (10) and (11), respectively, of Table 6.

[61] "The Welding of Aluminum", by the Aluminum Co. of America; "The Aluminum Industry—Aluminum Products and Their Fabrication", by Messrs. Edwards, Frary, and Jeffries; and "Gas Welding Aluminum and Its Alloys", by G. O. Hoglund, *The Sheet Metal Worker,* February, 1936.

TABLE 6.—EFFECT OF COMPOSITION, COLD WORK, AND HEAT TREATMENT UPON SPECIFIC GRAVITY AND ELECTRICAL AND THERMAL CONDUCTIVITY

Item No. (see Table 7)	Description	Aluminum content (percentages)	ALLOYING ELEMENTS* (PERCENTAGES)					Specific gravity	Weight, in pounds per cubic inch	Electrical conductivity (percentages of international copper standard)	Thermal conductivity at 100° C (centimeter-gram-second units)
			Copper	Silicon	Manganese	Magnesium	All other alloying elements				
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
	Electrical conductor	99.5	....	....	....	....	....	....	....	61	....
1	Annealed	99.0†‡	....	....	....	....	....	2.71	0.098	59	0.53
	Hard rolled									57	0.51
2	Annealed	98.75†	....	....	1.25	....	....	2.73	0.099	50	0.45
	Hard rolled									40	0.36
6	Annealed	95.0	4.0	....	0.5	0.5	....	2.79	0.101	45	0.40
	Heat treated									30	0.27
10	Annealed	93.85	4.5	0.8	0.8	....	0.05	2.80	0.101	50	0.45
	Heat treated									40	0.36
4	Annealed	97.25†	....	....	....	2.5	0.25	2.67	0.096	35	0.32
	Hard rolled									35	0.32
13	Annealed	97.80	....	0.7	....	1.25	0.25	2.69	0.097	45	0.40
	Heat treated									40	0.36

* Remainder, aluminum plus normal impurities. † "Common alloys". ‡ Minimum allowable.

Effect of Cold Working.—As with other common metals, cold working affects the mechanical properties of aluminum and all its alloys. All the various strength properties, stiffness, and endurance values are increased, and the plasticity, malleability, reduction in area, and elongation, are decreased. The modulus of elasticity is not affected.

For certain types of deforming operations, such as the cold rolling of sheets, in which the reduction in thickness is an accurate measure of the degree of cold working, a formula has been developed by R. L. Templin,[62] M. Am. Soc. C. E., which permits the prediction of the new tensile strength after the material has been reduced in thickness any specified amount. This formula is,

$$T = T_o\,(1 + 0.9\,R) \qquad \text{(2)}$$

in which T = the new tensile strength, in pounds per square inch; T_o = the tensile strength of the properly annealed metal; and R = the reduction in thickness expressed as a decimal. Aluminum products which have undergone an 80% reduction of cross-sectional area by cold working, without intermediate annealing, are considered "hard", and the rate of strain hardening up to this point may be considered constant. Equation (2) applies to cold working up to the equivalent of $R = 0.80$. Mr. Templin[63] also shows the relationship between the tensile strength and the yield strength, and offers a formula to calculate the new elongation. As these latter formulas are rather complex, and as their use is based upon certain limitations, too lengthy to include in this paper, the reader is referred to the quoted

[62] "The Aluminum Industry", Vol. II; "Aluminum Products and Their Fabrication", Edwards, Frary, and Jeffries, p. 406.

[63] *Loc. cit.*, p. 410.

reference. As a general rule the yield strength increases and the elongation decreases during the early stages of cold working at a much more rapid rate than the tensile strength increases. In annealed material, the yield strength is about one-third the ultimate tensile strength. With about 20% or more of cold working the yield strength is about 80 to 95% of the ultimate strength.

However, most of the forming methods are complex, or deform various portions of the metal in different amounts; for example, in forming a simple right-angle bend, only that portion of the metal approximately included within the bending arc becomes worked, and that to a variable extent. The fibers along the inside surface will have been subjected to compression, whereas those on the outer surface will have been subjected to stretching, both of which types of deformation affect the metal in the general manner just indicated. No formula has yet been devised which measures the magnitude of such working. When forming a shape in a draw press, the metal is stretched in a longitudinal direction and is deformed under compression in a transverse direction with usually no appreciable change in thickness. Furthermore, as the metal slides over the radius of the die, it is first bent 90° and immediately thereafter it is re-straightened. Such a complex deformation introduces a large amount of working. No formula exists, which predicts the degree of the cold work nor, quantitatively, the values of the new set of properties. Although certain problems would be simplified if such formulas were available, their non-existence is not a particularly serious handicap.

It is the fabricator's task to fashion the metal into the shape and design as specified without fracturing it. He must depend on his own past experience and that of others in like or similar cases, and to "cut and try" methods. If a sample specimen fractures upon attempting to form a certain bend, a larger forming radius must be used, or a softer temper, and perhaps the shaped part will have to be hardened by heat treatment. The manner in which the working is performed may be important at times; for example, when forming a rivet head, it is preferable to accomplish the deformation by one squeeze or by a few sufficiently heavy blows, rather than by a great many lighter blows. Under the action of one continuous squeeze, the metal is quite uniformly worked throughout its mass. With many light blows, a much larger percentage of the energy is first absorbed by the surface layers before the centrally located portions have become deformed at all. If carried to the extreme, such surface hardening may be the cause of starting surface cracks before a satisfactory head will have been formed. The principle in this case is that cold working hardens the material and gradually depletes its capacity to withstand additional cold deformation. The rate of this effect varies among the metals. Tin and lead harden but very slowly; copper, iron, and aluminum harden much more rapidly. The alloys of aluminum (and this is generally true for the other metals) harden more rapidly than the pure metal; that is, with large total amounts of cold deformation a re-softening (annealing) might be necessary with the alloy, whereas this might not be required with the pure metal.

The engineer, on the other hand, is interested mainly in this matter of work hardness, in terms of the capacity of the finished structure to perform

its intended purpose satisfactorily. This involves at least two questions: (1) Will the newly made structure possess adequate strength? and (2), will it retain that strength under the anticipated service conditions?

Because of lack of 100% accuracy of design methods, regardless of metal, influenced partly by the inability of any formula to take into consideration all the tolerances necessitated by commercial limitations, the answer to the first question is frequently determined by testing sample structures and accurately measuring the stresses induced in various members. With reference to the second question, it can be stated for aluminum and its alloys that the effects of cold work remain substantially constant unless some subsequent heating, sufficient in magnitude, should cause some change in the properties of the metal.

Cold work has little effect on the corrosion resistance of the aluminum alloys. As a matter of fact, there is some evidence that cold working may be slightly beneficial. Tests indicate that an alloy containing aluminum, copper, magnesium, and manganese (Item No. 4, Table 6) which has been heat treated and subsequently further reduced by cold working, is somewhat more resistant than the same alloy, heat treated but without the additional cold work. However, the magnitude of this difference is insufficient to give it commercial importance, other than to offer assurance as to the harmlessness of such cold work.

Cold work does not affect the weldability of any of the aluminum alloys. This would appear obvious as the heat, introduced by the welding operation, merely removes the effects of the cold working before the temperature of the metal has reached the point where welding can take place.

Cold work does not appreciably affect the specific gravity nor the thermal expansion. The electrical and thermal conductivity may be slightly reduced as indicated in Table 6.

Effect of Heating and Cooling.—When any aluminum alloy that has been merely cold worked is heated to its annealing temperature the effects of that cold work are removed immediately. This operation is called "annealing." Details of proper annealing practice for the several alloys are well covered by existing literature and will not be repeated herein. This softening effect is of importance to the fabricator as occasionally he must have recourse to annealing in order to re-soften the metal and thus permit of further cold deformation. Any of the aluminum alloys may be cold worked, annealed, and further cold worked any number of cycles desired. If the annealing is performed correctly, the product will possess properties identical with those of the annealed temper.

It is preferable to heat the material rapidly as this is conducive to producing a fine crystal structure—a coarse structure being somewhat weaker. The more rapid the heating, the finer the metal structure. This is more critical with small amounts of cold work (equivalent to about 4% reduction in thickness) than with larger amounts.

In the annealing of the "common alloys" (that is, those not susceptible to increasing strength by a "heat-treating" operation), no particular precaution

or care need be exercised against heating the metal somewhat above the required annealing temperature. Of course, heating the metal to, say, 800° F, when 650° F would have accomplished all that was desired, is unnecessary. It would be of value to have the furnace temperature substantially above the desired final metal temperature in order to re-crystallize quickly and thus obtain the finest crystal structure possible, but the load should be removed when the metal has reached the desired temperature. The feasibility of utilizing such a procedure depends on the particular furnace. Continuing to heat the metal beyond that point, and holding it there, is conducive to causing a coarser and, hence, less desirable crystal structure. Although no harm will result should the metal cool rapidly, such cooling is not generally of value. Something might be said in favor of permitting the mass to cool slowly on the grounds that the central portions of the load might not have reached the desired temperature, and such discrepancy would tend to be corrected by permitting a large mass to cool slowly.

With heat-treatable alloys, the principles are somewhat different. This can be better appreciated after covering the matter of heat treatment of aluminum alloys. No attempt will be made to describe in detail what occurs within the internal structure during and following the heat-treating operation. This also is well covered in the literature. Only a very brief résumé will be given herein.

The heat treatment of the aluminum alloys involves heating the metal to specified elevated temperatures, so as to put certain constituents in solution in amounts greater than their solubility at room temperature. Upon quenching the alloy, after such heating, it is in an unstable condition because of the excess of constituent in solution. This excess tends to precipitate, the effect being observable by an increase in strength, hardness, and reduction in general workability, but with no lowering of elongation values. In the case of certain alloys, such as the aluminum-copper-magnesium-manganese type, previously mentioned, this precipitation and self-hardening occur spontaneously at ordinary temperatures, very rapidly over the first few hours after quenching and gradually diminishing in intensity, substantially completing itself in about three or four days. The rate of this action may be retarded or completely postponed by placing the freshly quenched material in cold storage. Alloy 17S (Item No. 6, Table 7), when freshly quenched and placed in storage at 0°C, shows no precipitation after several days. Upon removing the material from cold storage and permitting it to warm again to ordinary temperatures, precipitation commences and continues in a normal manner. Advantage is taken of this delay in precipitation when it is desired to heat treat and quench a relatively large quantity of material at one time and maintain it in a soft and more workable condition for some days prior to forming the entire amount. Other compositions require a re-heating for several hours at temperatures approximating 300° F to cause this precipitation to occur to its maximum amount. The first heating and quenching cycle is termed "solution heat treatment," whereas the second type of heating is termed "precipitation heat treatment."

The foregoing indicates several principles of particular importance to the metal fabricator and the structural engineer. The highest strengths obtainable from these alloys result from heat treatment, although, of course, if the metal is subjected later to cold working the strengths may be increased still further. The fabricator frequently makes use of the fact that the metal is softer and more workable after the solution heat treatment, and before aging or precipitation has progressed to any substantial extent, by forming promptly after quenching. In certain more difficult cases, the metal may be formed in the annealed temper and the shape then heat treated.

Although each alloy has its most suitable heat treatment temperature, substantial solution of the constituent occurs below this optimum temperature and, in fact, the solution commences shortly above the annealing temperatures. Hence, when annealing the heat-treatable alloys, if fully annealed properties are desired, it is important to adhere as closely as possible to the recommended annealing temperature—namely, 650° F plus or minus 10 degrees. The rate of cooling is not important if these temperature limits have been observed. However, if these temperatures have been exceeded unintentionally, heat-treating effects may be avoided by slow cooling. As an extreme illustration, Alloy 17S (Item No. 6, Table 7), fully heat treated may be converted to the annealed condition by heating at 800° F for 2 hr and then cooling at a rate not exceeding 50° F per hr to less than 500° F. Occasionally, it is desired to produce some substantial softening, although without necessarily obtaining true annealed properties. In such an event, greater liberties may be taken with respect to temperatures and rate of cooling.

As the temperature of aluminum alloys is raised, the malleability and plasticity are markedly increased and the strength properties and resistance to deformation are decreased[64]. The metal fabricator may use this principle either to permit forming especially heavy sections, with less energy input than would be required by cold working, or to permit a severity of deformation that could not be accomplished cold without the metal fracturing. Even heating to as low as 400° F facilitates forming to a substantial extent. Occasionally, the material is heated to its proper heat-treating temperature and then, while still hot, pressed into the required shape, the cold dies producing substantial quenching effects[65].

There is not much difference in the resistance to corrosion between the several tempers of any common aluminum alloy. The word, "temper," in aluminum parlance, signifies any of the several metallurgical conditions in which wrought aluminum or its alloys is produced. For example, Item No. 2, Table 6 (containing 1.25% manganese, with the remainder of aluminum plus normal impurities) is produced in several "tempers:"

The soft temper is produced by annealing the alloy to remove the effects of cold working. The harder tempers are produced by strain hardening the

[64] "Properties of Wrought Aluminum Alloys at Elevated Temperatures", by F. M. Howell and D. A. Paul. *Metals and Alloys*, October, 1935.

[65] U. S. Patent 1 751 500.

alloy by varying amounts after it has been annealed. The alloy that has been cold worked the maximum amount practicable in commercial manufacturing operations, is said to be in the hard temper. By proper selection of the amount of strain hardening, the quarter-hard, half-hard, and three-quarter-hard tempers are produced, in which the tensile strengths are intermediate between those of the soft and the hard tempers in the manner indicated by the fractional designations. As these various tempers are merely the result of cold work, and as heating merely removes this cold-work effect, heating common alloys has no material effect on the resistance to corrosion. The facts are different with the heat-treatable alloys. In the case of Item No. 6, Table 6, the temper possessing the greatest resistance to corrosion is the heat-treated condition. Heating this material at temperatures much in excess of 212° F, for any substantial length of time, impairs its resistance. On the other hand, Item No. 13, Table 6, possesses an excellent and like corrosion resistance in any temper and, hence, heating any particular temper of this alloy produces no effect on its resistance. As a general rule, the heat-treatable alloys are in their most resistant state when they have been subjected to solution heat treatment. However, there are exceptions, as noted. Hence, when resistance to corrosion is of real importance, and it is intended to heat the material, the specific characteristics of the particular alloy should be ascertained and procedure governed accordingly. For example, the heat introduced by torch welding is sufficient, for the most part, to remove the heat-treatment effect, and so adversely affect the resistance to corrosion of a small area on either side of the weld in Item No. 6, Table 6 (heat-treated temper). On the other hand, in the case of Item No. 13, Table 6, no such harm results. The detailed facts are well known and are available either in present publications or from the producer of the alloy.

Whether or not the alloy may have been heated previously, has no effect upon its inherent weldability. However, in certain cases, a previous thermal treatment might increase the quantity of the oxide film on the surface of the metal and this, in turn, may affect the ease of welding. Upon heating Item No. 6, Table 6, to rather elevated temperatures, an oxide film is formed that interferes quite markedly with spot welding. For this reason, in order to spot weld or seam weld Item No. 6 (the heat-treated temper) the surface along the line of welding should be cleaned with emery cloth to remove this heavy oxide film. This is not necessary in the case of all compositions. This, again, is a matter of specific information which, however, is available.

Heating may affect certain other physical properties. When heating merely removes strain hardening, the effect upon electrical and thermal conductivity is just the reverse of introducing cold work, as previously mentioned. In the case of those alloys in which heating changes the degree of solution of certain constituents, the general rule is that, as the quantity of constituent put into solution increases, the electrical and thermal conductivity decreases. This is illustrated in Table 6. Heating and cooling have no appreciable effect on specific gravity.

Available Forms and Specific Properties of Structural Aluminum Alloys

Wrought Products.—Table 7 shows the nominal composition and the mechanical properties of those wrought alloys of interest in structural fields.

TABLE 7.—Nominal Composition and Typical Properties of Wrought-Aluminum Alloys

Item No.[a]	Alloy (1)	Description (2)	Aluminum content (percentages) (3)	Alloying Elements (Percentages): Copper (4)	Silicon (5)	Manganese (6)	Magnesium (7)	Chromium (8)	Nickel (9)	Zinc (10)	All other alloying elements (11)	Unit Stresses, in Kips per Square Inch: Tensile strength (12)	Yield strength (13)	Shear strength (14)	Endurance limit (15)	Percentage elongation[h] in 2 in. (16)	Brinell hardness number (17)
		(a) Common Alloys															
1	2 S	Annealed	99.0[e]	....	....	....	....	....	....	....	....	13	4	9.5	5.0	35	23
		Half hard[b]										17	14	11.0	7.0	9	32
		Hard rolled										24	21	13.0	8.5	5	44
2	3 S	Annealed	98.75	....	....	1.25	....	....	....	....	....	16	5	11.0	7.0	30	28
		Half hard[b]										21	18	14.0	9.0	8	40
		Hard rolled										29	25	16.0	10.0	4	55
3	4 S	Annealed	97.75	....	....	1.25	1.0	....	....	....	....	26	10	16.0	14.0	20	45
		Half hard[b]										35	31	19.0	15.0	5	65
		Hard rolled										42	38	23.0	16.0	3	80
4	52 S	Annealed	97.25	....	....	....	2.5	0.25	....	....	....	29	14	18.0	17.0	25	45
		Half hard[b]										37	29	21.0	19.0	10	67
		Hard rolled										41	36	24.0	20.5	7	85
		(b) Heat-Treatable Alloys															
5	14 S	Heat treated[c]	93.70	4.4	0.80	0.75	0.35	....	....	....	....	68	53	45	15.0	12[i]	130
6	17 S	Annealed	95.00	4.0	....	0.50	0.50	....	....	....	....	26	10	18	11.0	20	45
		Heat treated										58	35	35	15.0	20	100
7	A 17 S	Heat treated[d]	97.20	2.5	....	....	0.30	....	....	....	....	43	24	25	13.5	24	70
8	24 S	Annealed	93.80	4.2	....	0.50	1.50	....	....	....	....	26	10	18	14.0	20	42
		Heat treated										65	43	40	14.5	20	105
9	25 S	Heat treated[c]	93.90	4.5	0.80	0.80	....	....	....	....	....	58	35	35	15.0	18[i]	100
10	27 S	Heat treated	93.90	4.5	0.80	0.80	....	....	....	....	[f]	60	50	37	13.0	9	115
11	51 S	Annealed	98.40	....	1.00	....	0.60	....	....	....	....	16	6	11	6.5	30	28
		W										35	20	21	10.5	24	64
		Heat treated										48	38	30	10.5	14	95
12	A 51 S	Heat treated[c]	98.15	....	1.00	....	0.60	0.25	....	....	....	48	38	32	10.5	14[i]	90
13	53 S	Annealed	97.80	....	0.70	....	1.25	0.25	....	....	....	16	7	11	7.5	25	26
		W										33	20	22	10.0	22	65
		Heat treated										38	32	26	11.0	14	80
14	Ni[d] A	Heat treated	97.50	0.5	....	....	0.50	....	1.0	....	0.5[g]	47	40	....	13.5	9	....
15	Ni[j] X	Heat treated[d]	95.75	1.0	....	....	0.75	....	1.0	1.0	0.5[g]	51	41	....	13.5	16	....

[a] See corresponding Item Numbers in Table 6. [b] Cold worked after its last annealing sufficient to raise the ultimate tensile strength to a point half way between the annealed and the full hard tempers. [c] Forgings only. [d] Extruded shapes. [e] Minimum allowable. [f] Traces. [g] Equal proportions of manganese, chromium, and molybdenum. [h] Except as noted these values are for $\frac{1}{16}$-in. sheets. [i] These values are for 0.505-in. round bars. [j] Known commercially as Nicral.

In some cases these compositions and heat treatments are protected by patents. Column (3), Table 7, indicates that the remaining material, other than the alloying elements, is aluminum plus normal impurities, such as iron and silicon. For the specimens listed, Young's modulus of elasticity is approximately 10 300 kips per sq in. Column (13), Table 7, contains the yield-strength stress that produces a permanent set of 0.2% of the initial gage length[66].

Referring to Column (16), Table 7, it is to be noted that the elongation will vary somewhat with the form and size of the specimen. The Brinell hardness test (Column (17), Table 7) involved a 500-kg load, exerted on a 10-mm ball. The values in Column (14) are single shear values, computed from double shear tests. The endurance limit (Column (15), Table 7) is based on a 500 000 000-cycle of complete reversed stress, using a standard form of machine and specimen[67].

There are well developed and standardized procedures for determining the mechanical properties and other characteristics of metals and there are well defined and accepted units of measurement by which these properties may be expressed[68]. On the other hand, in the case of resistance to corrosion, no such unit of measurement exists. This quality is evaluated by various empirical tests co-ordinated with actual field performance[69]. These observations merely indicate how one metal, or an alloy of a metal, behaves under certain corrosive conditions on a relative basis with another. Thus, the resistance to corrosion of a certain grade of aluminum cannot be given a rating of D, whereas another grade of another metal would be assigned a rating of E (in which the letters are supposed to possess definite numerical value). One can only state that the first is "somewhat superior" or "very much superior" or "inferior" to the other. After due experience of course, one can state, correctly, that Metal A will perform satisfactorily in a certain service, whereas Metal B will not. An attempt to list the relative resistance of the various products in tabular form would be misleading, as any correct rating should take into consideration the various forms in which the alloys are available and the service conditions applicable in any case. Although the resistance to corrosion of Item No. 9, Table 7, heat-treated, is not as great as that of Item No. 8, similarly treated, the former is utilized primarily as forgings, relatively thick in section, whereas the latter, which is not available as forgings, is normally used in thin sections, and usually under far more severely corrosive conditions. The resistance to corrosion of forgings made of Item No. 9, has proved quite adequate in the fields where they are adopted. The matter, therefore, can best be treated by discussion rather than by any table.

In general, it can be stated that aluminum alloys belong to the class which may properly be described as metals resistant to corrosion. Items Nos. 1, 2, 4,

---

[66] Specification E8–33, Am. Soc. for Testing Materials.

[67] "Notes on Fatigue Tests on Rotating-Beam Testing Machines", Appendix to Rept. of Research Committee on Fatigue of Metals, *Proceedings*, A. S. T. M. Vol. 35, Pt. 1, 1935.

[68] "Standard Methods for Tension Testing of Metallic Materials", Am. Soc. for Testing Materials (E8–33).

[69] "Corrosion Resistance of Structural Aluminum", by E. H. Dix, Jr., *Proceedings*, Am. Soc. for Testing Materials, Vol. 33, Pt. II, 1933.

and 13, Tables 6 and 7; and Item No. 3, Table 7, in all tempers, possess superior resistance. In fact, except under the most severe service, such as the marine, they usually need not be painted for protection. Referring to Table 7, Items Nos. 5, 6, 7, 8, 10, and 14 (heat treated) and Item No. 11 (in all tempers), are next in order and substantially equal. Normally, they should be painted, although, in relatively heavy sections, such as would generally be used with forgings of Item No. 5 (heat treated), this might not be necessary. Next, would come Item No. 15, Item No. 9 (heat treated), and Items Nos. 6 and 8 (annealed). The most satisfactory way for the prospective user to take care of this subject is to consult with the manufacturer who is to supply the material.

Aluminum alloys are available in the form of sheets, plates, wire, rods and bars, rolled and extruded shapes, tubing, rivets, and forgings, although not all the compositions listed in Table 7 are available in all these forms. Table 8 shows those forms that to-day are classed as "standard"; that is,

TABLE 8.—STANDARD COMMODITIES WROUGHT ALLOYS
(Commodities marked with asterisk are standard).

Item No. (same as in Table 7)	Alloy	Sheets	Plates	Wires	Rod and bars	Rolled shapes	Extruded shapes	Tubing and pipe	Rivets	Forgings
1......	2 S	*	*	*	*	..	*	*	*	..
2......	3 S	*	*	*	*	..	*	*	*	..
3......	4 S	*	*	..	..	..	..	..	..	..
5......	14 S	..	..	..	..	..	..	..	..	*
6......	17 S	*	*	*	*	*	*	*	*	*
7......	A 17 S	..	..	..	..	..	..	..	*	..
8......	24 S	*	..	..	..	..	*	*	..	..
9......	25 S	..	..	..	..	..	..	..	..	*
10......	27 S	*	*	..	..	..	..	..	..	..
11......	51 S	*	*	*	*	..	*	*	..	..
12......	A 51 S	..	..	..	..	..	..	..	..	*
4......	52 S	*	*	*	*	..	..	*	..	..
13......	53 S	*	*	*	*	..	*	*	*	*
14......	Ni A	*	..	..	..	..	..	..	..	..
15......	Ni X	..	..	..	..	..	*	..	..	..

are regularly produced in routine commercial production. Exceptions are made when warranted; for instance, although Item No. 6 (heat treated), is the only composition regularly carried in stock in the form of rolled shapes, Item No. 10 (heat treated), has been produced in rolled shapes when the quantities required, have justified it.

Plates are being produced as heavy as 2 000 lb. Although there are limitations as to sizes of certain alloys and tempers, they are available in widths of as much as 120 in. and lengths as determined by the 2 000-lb weight. Heat-treated rolled shapes in various sections and sizes up to 12-in. channels are produced in lengths as great as 85 ft. Heat-treated extruded shapes are available in lengths of about 50 ft.

Item No. 6, Table 8, (17S) is available in all the various forms, and because it presents an unusually favorable combination of properties, it has ranked as the most generally useful of the strong alloys.

The particular merits of Item No. 10 (heat treated) (27S-T) are its relatively high yield strength and the fact that, as this temper is produced by aging for some hours at approximately 300° F, neither the mechanical properties nor the corrosion resistance is impaired should the material be re-heated to this temperature, or even to slightly higher temperatures. This behavior is of significance because it permits the use of hot-driven steel rivets in joining heat-treated members composed of this alloy, without fear that the heat, thus introduced, will adversely affect the properties of the aluminum parts. The most noteworthy applications of Item No. 10 (heat treated) to date, comprise the rehabilitation of the floor of the Smithfield Street Bridge[70], Pittsburgh, Pa.; the floor of the Stratford Avenue Bridge, Bridgeport, Conn.; the side walk and railing of the Covington-Cincinnati Suspension Bridge[71], across the Ohio River; and the railings on the Laredo Bridge, in Texas. A thorough study has been made, contemplating the use of aluminum for the trusses and floor of the famous Brooklyn (N. Y.) Suspension Bridge[72].

The corrosion resistance of Item No. 13, Table 8, (53S) which is of a high order, is substantially the same in all tempers. It is for this reason that this alloy has found wide use in the construction of sewage disposal plants, in the form of extruded sections in the architectural industry for the fabrication of windows and frames, doors, building fronts, and various ornamental constructions. Increasingly, such items have been treated by a patented process, which applies a hard, weather-resistant, and corrosion-resistant coating and one that facilitates cleaning.

Cast Products.—Table 9 gives the composition of castings that are of widest interest to the structural engineer and shows their typical mechanical properties.

As will be noted, the heat-treated, aluminum-copper (Item No. 23, Table 9) and aluminum magnesium (Item No. 24), alloys are the strongest. Item No. 23, Table 9, has been used rather extensively since about 1920, whereas the alloy (Item No. 24) is a more recent development and still requires special handling in the foundry. The latter represents the highest combination of strength and shock resistance yet attained in aluminum castings and is especially desirable for large sections. It has given good service performance as parts of large shovel dippers. Fig. 27 shows the back of a 7-cu yd dipper, 6.83 ft long, 4.90 ft wide, and 3.0 ft deep, weighing 2 326 lb. Fig. 28 shows one hinge for a 32-cu yd dipper, 12.58 ft long, 5.25 ft wide, and 1.50 ft deep, weighing 2 242 lb. These castings are of Alloy 220 (Item No. 24, Table 9).

[70] "Aluminum Enters Bridge Construction", by Charles M. Reppert, M. Am. Soc. C. E., *Engineering News-Record,* November 9, 1933; "Erecting an Aluminum Bridge Floor", by Henry D. Johnson, Jr., Assoc. M. Am. Soc. C. E., *Engineering News-Record,* November 23, 1933; and "Fabricating Structural Aluminum", by C. G. Schade, M. Am. Soc. C. E., *Engineering News-Record,* December 28, 1933.

[71] "The Use of Structural Aluminum in Bridges", by J. P. Growdon, M. Am. Soc. C. E., *Bulletin,* Associated State Eng. Societies, April, 1935.

[72] "Aluminum Trusses and Floor for Brooklyn Bridge", *Engineering News-Record,* April 18, 1935.

TABLE 9.—COMPOSITION AND MECHANICAL PROPERTIES OF ALUMINUM CASTING ALLOYS*

Item No.	Alloy	Remarks	Aluminum content † (percentage)	Alloying Elements (Percentages)					Unit Stresses, in Kips per Square Inch				Percentage elongation in 2 in.	Brinell hardness number
				Copper	Iron	Silicon	Magnesium	Zinc	Tensile strength	Yield strength	Shear strength	Endurance limit		
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
						(a) Common Alloys								
16	43	As cast........	95.0	....	....	5.0	.....	....	19	9	15.0	6.5	4.0	40
17	47	As cast........	87.5	....	....	12.5	.....	....	26	11	18.0	6.0	8.0	50
18	112	As cast........	89.8	7.5	1.2	....	.....	1.5	22	14	20.0	7.5	2.0	65
19	212	As cast........	89.8	8.0	1.0	1.2	.....	....	22	14	20.0	8.5	2.0	70
20	214	As cast........	96.25	....	....	....	3.75	....	25	12	19.0	5.5	9.0	50
21	216	As cast........	94.00	....	....	....	6.00	....	27	16	23.5	.....	6.0	60
						(b) Heat-Treated Alloys								
22	195	Heat treated...	96.0	4.0	....	....	.....	....	31	16	28.0	6.0	8.0	65
23	195‡	Heat treated...	96.0	4.0	....	....	.....	....	36	22	30.0	6.5	4.0	80
24	220	Heat treated...	90.00	....	....	....	10.00	....	44	26	33.5	7.5	13.0	75
25	355	Heat treated...	93.25	1.25	....	5.0	0.50	....	30	20	30.0	.....	4.0	60
26	355‡	Heat treated...	93.25	1.25	....	5.0	0.50	....	35	27	30.0	.....	2.0	80
27	356	Heat treatment	92.70	....	....	7.0	0.30	....	28	16	22.0	.....	6.0	55
28	356‡	Heat treatment	92.70	....	....	7.0	0.30	....	32	22	22.0	8.0	4.0	70

* In the case of some of these alloys, the composition or heat treatment, or both composition and heat treatment are patented.

† Plus normal impurities, such as iron and silicon.

‡ Different type of heat treatment.

This should be considered a special alloy for unusually severe service. The alloy (Item No. 20) is used where service conditions require the maxim resistance to corrosive attack. It is more difficult to cast into intricate, leak-proof castings than the aluminum-silicon alloys, but has higher mechanical properties than

FIG. 27.—ONE HINGE FOR 32-CUBIC YARD DIPPER.

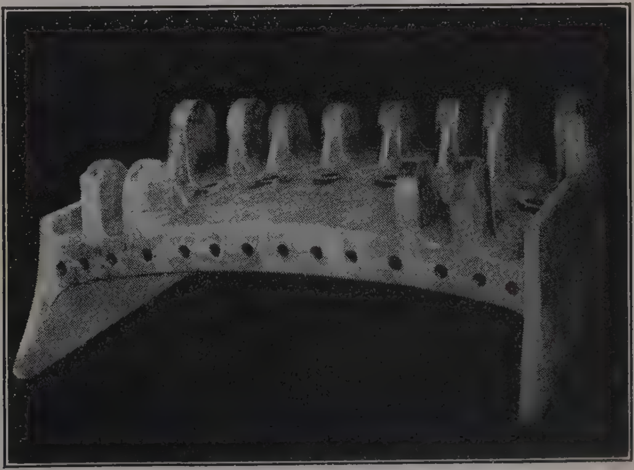


FIG. 28.—BACK FOR 7-CUBIC YARD DIPPER.

the alloy listed as Item No. 16, in Table 9, and is distinctly more resistant to corrosion. For this reason, it is utilized in the production of castings for use in sewage disposal plants, and for marine applications. The alloy (Item No. 21) is in a somewhat similar category, with equal resistance to corrosion, higher tensile and yield strengths, but with somewhat lower elongation.

The founding of aluminum alloys differs in many respects from that pertaining to other metals; in fact, each metal possesses its own foundry characteristics. It is important, therefore, when considering the detail design of any particular casting, and especially the pattern, to consult with the foundryman in order to permit of certain small modifications in design that might enable simplification of foundry problems, with consequent reduction in cost, that will insure the highest and greatest uniformity of properties.

The size and complexity of castings that can be produced varies so much with the particular instance that no general specifications can be given.

Metallurgical Aspects of Fabricating Structural Aluminum Alloys

Aluminum alloys are almost always cut by mechanical means, such as shearing, sawing, or machining. Torch cutting is seldom recommended because of the high heat conductivity which makes this method impractical. Furthermore, the heat introduced would have adverse effects in certain instances, as previously discussed.

Aluminum alloys are formed by all the common methods, although they are usually done cold. They are occasionally formed hot for heavy sections and for forging, although consideration should be given as to whether the degree of heating has adversely affected strength or resistance to corrosion, and whether the shaped part should be re-heat treated.

Joints are fabricated by torch, electric spot, seam, arc, and butt welding, and by riveting and bolting. The choice of method depends upon the results desired and as influenced by the effect of heat upon the particular alloy or temper being used.

When using rivets larger than $\frac{3}{8}$ in. in diameter, the present usual practice is to use hot steel rivets. Due to the high heat conductivity of aluminum, the heat thus introduced is rapidly dissipated with small chance of harmful results, especially when the rivets are driven in a scattered manner. In sizes smaller than $\frac{3}{8}$ in., rivets are driven cold and, in the case of Alloy 17S (Item No. 6, Table 7), promptly after quenching before age hardening has progressed to any extent. Use is also made of the fact that this aging is arrested by placing the freshly quenched rivets in cold storage as previously mentioned. This process avoids the necessity of co-ordinating the heat treating and driving operations.

Torch welding requires the use of a flux, which should be removed later by a thorough washing to prevent a chemical attack of the metal.

Because of the high thermal and electrical conductivity of aluminum, the ordinary electric welder is not suitable. Suitable spot and seam-welding equipment has been developed, which can be relied upon, consistently, to reproduce spots, or a seam, of high quality. Although the quantity of heat introduced by the electric method is substantially less than that arising in

torch welding, the effect of such heat should be considered. Arc welding is also being developed rapidly and is beginning to find commercial application. In this case, also, the amount of heating is substantially less than with torch welding, and harmful or annoying effects upon resistance to corrosion, or distortion, are greatly minimized. In general, torch welds of common alloys should be designed on the basis of the annealed temper. Although the heating does not fully anneal the heat-treated materials, there will be some softening and some over-aging in a small area on either side of the weld, thus reducing the strength and usually the resistance to corrosion. This effect can be corrected partly by a re-heat treatment. Usually, therefore, the strong alloys are not torch welded. The heating effects by the electric method are sufficiently less than by other methods so that it is coming into increasing use, especially in lighter work.

PART II.—MAGNESIUM

Historical

Although magnesium was isolated as an element as early as 1830, production of the metal on a commercial scale was not attempted until 1900 so that it can be truly called a product of the Twentieth Century. Historically, it was first produced about 1808 as a mercury amalgam by Sir Humphrey Davy who reduced the heated oxide with potassium vapor. In 1830, Bussy, the French chemist, obtained a fairly pure product by reducing magnesium chloride with potassium. In 1856, Deville and Caron used sodium for the reduction of the chloride followed by distillation in hydrogen, but the resulting metal was impure because of the presence of sodium and air reaction products. In 1863, Sonstadt developed a process in England based on the reduction of the anhydrous chloride by sodium, and a company was organized for small quantity production.

Commercial success was slow and the industry was of little importance until the electrolytic process was developed in Germany in the early part of the century. The development of fabricated products, such as sheets, castings, extrusions, and rods was undertaken and added considerably to the uses for the metal. The metal and its alloys were made and marketed under the name, Elektron, and, to-day, in England and on the Continent, this trade-mark is synonymous for magnesium alloys. Interest in the production of magnesium in the United States came in 1915 when the World War prevented importation from abroad.

General Metallurgical Principles of Structural Magnesium Alloys

Effect of Composition.—At present, most of the magnesium base alloys used in the United States, for structural purposes, are either ternary magnesium-aluminum-manganese or quaternary magnesium-aluminum-zinc-manganese alloys. The principal exception is the binary magnesium-manganese alloy which is used, preferably in the wrought condition, in applications where resistance to corrosion and ease of forming and welding are the chief requirements.

All the magnesium alloys now used for engineering purposes contain manganese. The binary magnesium-manganese alloys, usually containing 1.2 to 2.0% manganese, possess the greatest resistance to atmospheric and salt-water corrosion of any of the known magnesium base alloys, and manganese is added to the alloys containing aluminum, or aluminum and zinc, to enhance their resistance to corrosion. The manganese content of these alloys is rather definitely limited by the quantity of aluminum present and the possible variations are without influence on the more commonly determined physical properties. There is some evidence, however, that the addition of manganese to a wrought binary magnesium-aluminum alloy causes a marked increase in the proportional limit.

At present, aluminum is the most important element added to magnesium to produce structurally useful alloys. Alloys in commercial use in the United States contain aluminum in quantities varying from 3 to 13 per cent. Additions of aluminum in this range cause a marked increase in mechanical properties in both the cast and the wrought conditions.

Increasing the aluminum content of magnesium alloys causes changes in density, thermal conductivity, and electrical resistivity; but, in quantities up to 12% at least, the coefficient of thermal expansion is not greatly affected. The mean coefficient of expansion, α, for magnesium alloys from 0°C to any temperature, t, may be obtained from the following equation[63]:

$$\alpha = (25.07 + 0.00936\, t)\, 10^{-6} \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The changes in density, thermal conductivity, electrical resistivity, tensile properties, and hardness brought about by the addition of increasing amounts of aluminum to magnesium are shown in Figs. 29 to 32, inclusive. In the sand-cast alloys, still other combinations of properties than those shown in the graphs may be obtained by aging heat-treated alloys containing more than about 8% aluminum.

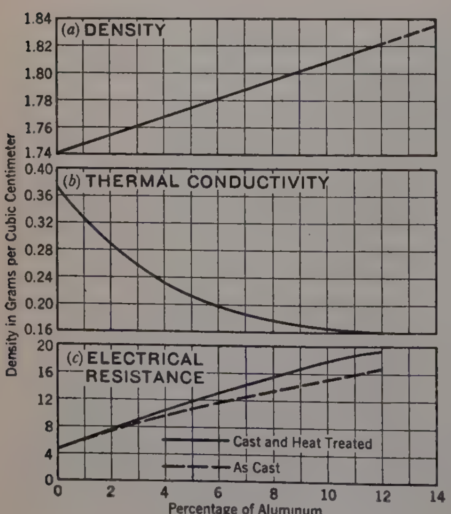


FIG. 29.

The shear strength and the compressive strength of magnesium-aluminum alloys tend to increase with the aluminum content in both the cast and the wrought conditions although this relation is more regular in the case of the wrought alloys. The shear strength of extruded pure magnesium is 16 kips per sq in. Additions of aluminum increase this value to a maximum of about 26 kips per sq in. in alloys containing 12% aluminum.

Additions of aluminum and zinc (either one or both) increase the endurance limit of magnesium

alloys at least up to the point where the combined content of alloying elements amounts to 10 or 11%; beyond this amount, reliable data on endurance limit are lacking. Numerical data as to the endurance limits of magnesium alloys are given in Table 10.

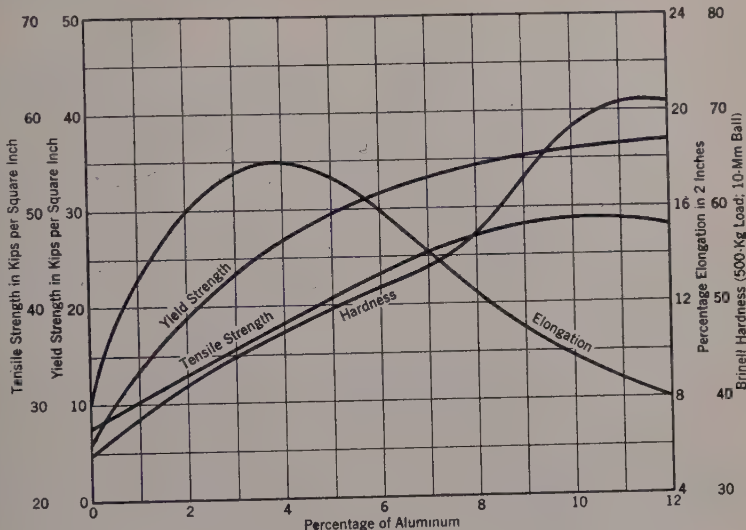


FIG. 30.—EXTRUDED ALLOYS.

The form in which these alloys are usually fabricated is as follows: Extrusions, Items Nos. 29, 30, 31, 32, and 33; sheets, Items Nos. 29 and 30; die castings, Item No. 38; sand and permanent mold castings, Items Nos. 34,

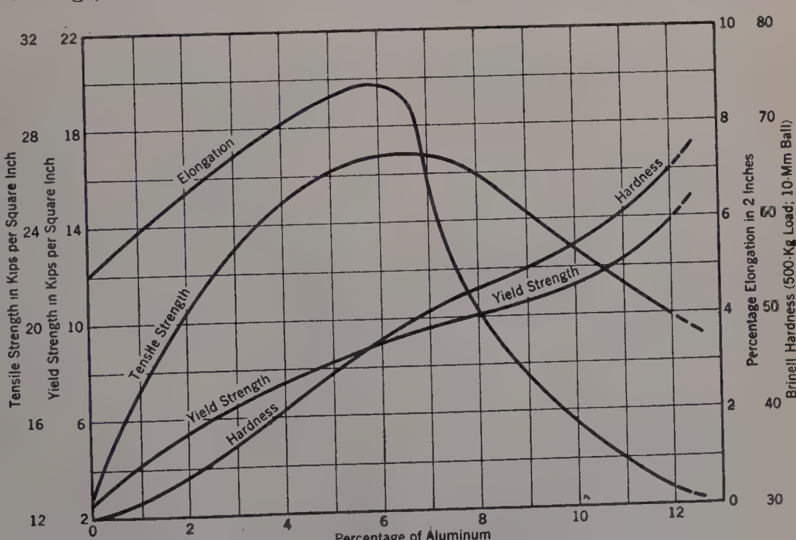


FIG. 31.—SAND CAST ALLOYS.

35, and 36; and, sand and semi-permanent mold castings, Item No. 37. Column (8), Table 10, gives the total allowable impurities, of which copper and nickel may not exceed the amount indicated in the starred footnote.

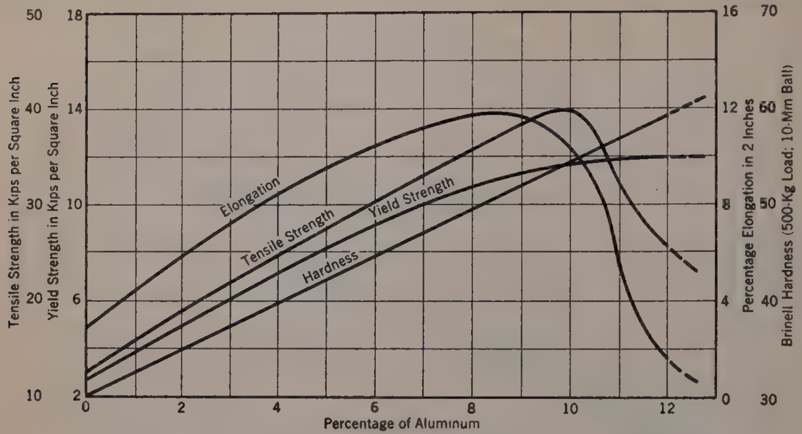


FIG. 32.—SAND CAST ALLOYS AFTER SOLUTION HEAT TREATMENT.

The yield point stress (see Column (10), Table 10) is defined as that which produces a permanent set of 0.2% of the initial gage length[66]. In Column (11), the shear strengths are single values derived from double shear

TABLE 10.—NOMINAL COMPOSITION AND TYPICAL MECHANICAL PROPERTIES OF MAGNESIUM ALLOYS

Item No.	Alloy, AM: (1)	Description (2)	Magnesium content (percentages) (3)	Alloying Elements (Percentages): Aluminum (4)	Manganese (5)	Zinc (6)	Silicon (7)	Other elements* (8)	Unit Stresses, in Kips per Square Inch: Tensile strength (9)	Yield strength (10)	Shear strength (11)	Endurance limit (12)	Percentage elongation in 2 in. (13)	Brinell hardness number (14)	Specific gravity (15)
29	3S	Soft sheet, heat treated †	97.9	....	1.5	.....	0.3‡	0.3‡	32	15	17.5	8.0	16.0	40	1.76
		Hard rolled sheet†.....							36	29	....	....	10.0	52	1.76
30	53S	Soft sheet, annealed†	94.6	4.0	0.3	0.3‡	0.5‡	0.3‡	36	22	18.5	10.0	18.0	50	1.77
		Hard rolled sheet†.....							44	35	....	....	10.0	60	1.77
31	57S	Extruded................	91.75	6.5	0.2	0.75	0.5‡	0.3‡	43	33	21.0	15.0	16.0	55	1.80
32	58S	Extruded................	90.00	8.5	0.2	0.5	0.5‡	0.3‡	46	35	22.0	16.0	13.0	60	1.81
33	59S	Extruded...............	89.10	10.0	0.1	.....	0.5‡	0.3‡	50	38	22.5	16.0	10.0	70	1.81
		Extruded, heat treated †.							56	40	25.5	16.0	3.5	85	1.81
34	240	Sand cast, heat treated†..	88.80	10.0	0.1	0.3‡	0.5‡	0.3‡	35	12	....	11.0	9.0	52	1.81
		Sand cast, heat treated, aged................							36	19	....	8.0	1.0	69	1.81
35	241	Sand cast, heat treated....	90.7	8.0	0.2	0.2‡	0.5‡	0.3‡	33	11	....	7.5	10.0	48	1.80
36	246	Sand cast, heat treated, aged	86.8	12.0	0.1	0.3‡	0.5‡	0.3‡	32	20	....	....	0.5	85	1.82
37	265	Sand cast...............	90.00	6.0	0.2	3.0	0.5‡	0.3‡	27	11	....	11.0	6.0	48	1.84
		Sand cast, heat treated...							35	12		11.0	10.0	50	1.84
		Sand cast, heat treated, aged................							37	19	....	9.0	4.0	69	1.84
38	230	Die cast................	88.85	10.0	0.1	0.3‡	0.75	0.3‡	28	22	....	....	0.5	63	.....

* Such as copper (maximum allowable, 0.05%) and nickel (maximum allowable, 0.03%) † Wrought materia
Maximum allowable

tests. The Brinell hardness number was based upon a 500-kg load on a 10-mm ball; and the endurance limits (Column (12), Table 10) were based on 500 000 000 cycles of stress. Composition has little influence on the modulus of elasticity. An average value suitable for design purposes is 6 500 kips per sq in.

The effect of aluminum concentration on the resistance to corrosion of magnesium-aluminum-manganese alloys depends to a considerable extent upon the microstructure. If the alloys are homogeneous that is, if all the aluminum is in solid solution, the corrosion resistance will decrease with increasing aluminum content, but if the alloys are heterogeneous, with considerable undissolved Mg_3Al_2 compound in the grain boundaries, the corrosion resistance will increase with aluminum content.

Torch welding of the magnesium-aluminum-manganese alloys becomes more difficult as the percentage of aluminum is increased. Alloys containing 4 to 5% of aluminum are welded without difficulty, but as the aluminum concentration is increased to more than 5%, hot shortness and cracking become gradually more serious. Torch welding of alloys containing more than about 8% aluminum is seldom recommended.

The alloys of magnesium used for structural purposes in the United States at present do not contain more than 3.5% zinc, and alloys with as much zinc as this are used only for casting. Zinc tends to make the alloys hot-short and for this reason the wrought alloys usually contain not more than 1% of this element. Like aluminum, zinc forms solid solutions with magnesium, and variations in zinc content are capable of producing changes in the mechanical and physical properties of the alloys.

The addition of zinc to binary alloys of magnesium and aluminum causes them to become more difficult to hot work and makes them less weldable. On the other hand, the addition of zinc usually improves the corrosion resistance and is slightly more effective than the equivalent percentage of aluminum in raising the tensile strength, yield strength, and hardness.

In the case of the magnesium-aluminum-zinc-manganese alloy used in the form of castings and containing a nominal 6% aluminum and 3% zinc, the zinc enters solid solution only after heat treatment and the changes in mechanical properties brought about by the zinc in this alloy are greater than can be obtained by adding an equivalent percentage of aluminum. In the sand cast condition, the 6% aluminum, 3% zinc alloy is stronger, more ductile, and tougher, and has a higher endurance limit than an alloy containing 9% aluminum. In the homogeneous heat-treated condition, the difference in mechanical properties is not marked, but the resistance to corrosion of the zinc-bearing alloy is very much superior to the 9% aluminum alloy. When these two alloys are artificially aged to enhance the yield strength and hardness, the beneficial influence of the zinc is again evidenced by greater strength, toughness, ductility, yield strength, and endurance limit combined with better resistance to corrosion than can be produced in the 9% aluminum alloy.

Although it is possible to weld alloys containing appreciable quantities of zinc, it becomes more difficult with increasing quantities. When the zinc content of wrought material is greater than 1%, or when the aluminum + zinc

content is 9% or more, hot-shortness and cracking become so serious that welding is not usually recommended.

Effect of Cold Working.—Magnesium and all magnesium alloys work harden very rapidly and, in general, the rapidity of work hardening increases with the quantity of alloying material present. Pure magnesium and the binary magnesium-manganese alloy sheets may be cold rolled about 25% before failure occurs, but magnesium-aluminum-manganese alloy sheets containing 4% aluminum harden more rapidly and are seldom cold rolled more than 15 per cent. Magnesium alloys are so refractory toward cold deformation that practically all wrought forms are produced by hot working and, for the same reason, practically all forming and shaping operations on magnesium alloy sheets, plates, tubing, or structural shapes are performed at elevated temperatures in the range from 225° to 350° C (437° to 662° F).

In common with other metals, magnesium alloys with sufficient plasticity to withstand cold working may be strengthened and hardened by this process. The effect on the mechanical properties may be seen by referring to Table 10, which gives the properties of sheets in the soft and hard tempers. It has also been found that cold working extruded alloys slightly (1%) by stretching has a beneficial effect on their properties. The stretching operation increases the tensile and yield strengths without reducing the ductility.

Cold-worked sheets usually have a slightly lower resistance to corrosion than annealed sheets of the same composition.

Effect of Heating and Cooling.—A comprehensive discussion of the effect of heating and cooling on magnesium alloys is impossible in the limited space of this paper. The effect of heating varies with the composition of the alloy, its previous history, the temperature to which it is heated, and the length of time it is held at high temperature.

When cold-worked magnesium alloys are heated, they become re-crystallized and consequently soft, and they lose tensile strength, yield strength, and hardness. This type of heating is termed annealing and a common method of annealing cold-worked magnesium is to bring the mass of metal to 650° F and allow it to cool in air. Sudden cooling of annealed material does no harm, but would serve no useful purpose. Hot-worked magnesium alloys, such as extrusions and forgings, usually have the best mechanical properties in the "as worked" condition and annealing tends to reduce the tensile and yield strengths without increasing the ductility. Wrought material is usually formed at temperatures that will anneal the metal and, for this reason, the heating to forming temperature should be as rapid as possible and the time at temperature should be curtailed. Annealing of magnesium-alloy castings is without benefit and, in many instances, will result in lower properties.

Magnesium alloys are amenable to heat treatments of the same type as those used for aluminum alloys as described in Part I; that is, a supersaturated solid solution of the alloying element or elements is effected by heating to a relatively high temperature followed by rapid cooling. This produces a structure that is substantially homogeneous, with certain characteristic mechanical properties associated with such a structure. If the maximum obtainable yield strength and hardness are desired, the solution heat treat-

ment is followed by heating the quenched (homogeneous) material for various times at lower temperatures, usually 12 to 20 hr at 300° to 350° F. This second heating or "artificial aging" is necessary because magnesium alloys do not age-harden at room temperature to any appreciable extent. The heat-treating temperature necessary to make them homogeneous depends upon the alloying constituent. Zinc may be put into solid solution at about 630° F, whereas 750° to 810° F is frequently used to effect the solution of the aluminum constituent.

As mentioned previously, wrought magnesium alloys generally possess their best mechanical properties in the as-worked condition and, for this reason, wrought alloys are seldom heat treated. In the case of magnesium alloy castings, however, heat treatment produces such a marked improvement in properties that the great majority of the castings supplied to the trade in the United States are in the heat-treated condition. The effect of heat treatment on the mechanical properties of cast magnesium alloys is shown in Table 10.

Heating and cooling have no effect on the welding characteristics of magnesium alloys nor on their specific gravity except that heat-treated and quenched alloys high in alloy content tend to "grow" very slightly if used at elevated temperatures. In common with aluminum alloys in which heating changes the degree of solid solution, the general rule is that, as the quantity of constituent put into solid solution increases, the electrical and thermal conductivity decreases.

The effect of temperatures encountered under service conditions must be taken into account when using magnesium alloys for engineering purposes. Although sub-normal temperatures as low as — 112° F have little or no effect on the mechanical properties, they undergo appreciable change at temperatures of approximately 300° F and more. Considerable data as to the effect of elevated temperatures on the properties of magnesium alloys are available and the one who is to supply the material should be consulted before magnesium is utilized in structures operating at elevated temperatures.

Available Forms and Specific Properties of Structural Magnesium Alloys

Wrought magnesium alloys are available as extrusions, sheets, plates, and forgings. Bars, rods, and structural shapes are produced by the extrusion process because of the readiness with which magnesium alloys may be worked in this manner. Rolled shapes and sections in high-strength magnesium alloys are not available at present. Because of the equipment and ingot sizes available, present extruded sections are limited in weight to about 300 lb and in size to sections corresponding to a standard 6-in. channel or I-beam. Rough press forgings are limited in weight to about 200 lb, whereas die forgings produced by hammering are limited to the forming capacity of an 18 000-lb hammer. Under present conditions obtaining in commercial fabricating plants, plates or sheets are limited in weight to about 80 lb, with a maximum width of 60 in. and a maximum length of 240 in.

Cast magnesium alloys are available as sand castings, permanent mould castings, and die castings. There are alloys readily adaptable for fabriction by all three methods, and the mechanical properties obtained compare favorably with aluminum alloy castings used for similar purposes. Particular atten-

tion is called to the high endurance limit of magnesium alloy castings as compared with most aluminum-base alloy castings. It should be mentioned, however, that sharp corners, notches, and other "stress raisers" seriously reduce the endurance limit of magnesium alloys, and this fact must be kept in mind when using such alloys in stressed structures.

Magnesium die castings and permanent mould castings are limited in size only by the die or mould equipment available. There is no inherent reason why magnesium castings of this type cannot be produced in the same sizes as die and permanent mould castings in other metals. Magnesium sand castings require a more specialized technique and oxidation inhibitors must be used in the moulding sand. In the present state of the art, sand castings with sections thicker than about 6 in., or which require more than about 1 500 lb of metal to pour would be difficult to produce. The composition of magnesium alloys is shown in Table 10.

Metallurgical Aspects of Fabrication of Structural Magnesium Alloys

Magnesium alloys may be fabricated into structural units or assemblies by machining, cutting, forming, welding, or riveting. The precautions to be observed when machining or cutting consist mainly in keeping the tool sharp and free from chips to avoid overheating and consequent firing of the chips. Cutting must always be done mechanically as the high heat conductivity of the metal and the tendency of molten magnesium to burn precludes the successful use of torch-cutting. If it is necessary to use a liquid coolant (magnesium alloys are usually machined without lubricant or coolant), one should be selected that will not tarnish the finished work. Mineral oils are usually satisfactory, but rancid animal oils or acidic soluble oils should be avoided.

Severe cold-forming operations should be avoided as they are likely to cause invisible cracks which, later, will cause failure. Hot forming must be performed at temperatures below the eutectic temperature for all alloys containing aluminum and zinc (either or both). The binary magnesium-manganese alloy may be heated to 450° C (842° F), if necessary, but usually 260° to 350° C (500° to 662° F) is hot enough. The wrought magnesium-aluminum-manganese or magnesium-aluminum-zinc-manganese alloys may be readily formed in the range, 260° to 350° C (500° to 662° F), and the alteration in mechanical properties caused by this heating is not important unless the heating is prolonged.

Magnesium alloys not too high in alloy content may be joined by torch welding and the various types of electric resistance welding known as spot, seam, and butt welding. At or near the weld, the structure of wrought materials reverts to that of a casting with consequent lowering of properties. This condition can be greatly improved by hammering the finished weld at temperatures in the range, 290° to 370° C (550° to 700° F), and with certain types of mechanically operated electric butt welders, a cast structure at or near the weld is avoided for the most part. In order to prevent corrosion, welding flux must be thoroughly removed from torch welds, and seam or spot welds should be freed from copper particles picked up from electrode tips.

Because of the danger of electrolytic corrosion (magnesium is electro-positive to all common structural metals), rivets for magnesium alloy structures

must be selected with care and, where possible, should be insulated with bitumastic paint. Magnesium alloy rivets cannot be used successfully because they would have a low shear strength and because even small sizes would have to be driven hot. Rivets of steel, copper, nickel, and copper-bearing aluminum alloys should be avoided. For joints not subject to high stresses, rivets of commercially pure aluminum (Item No. 1, Table 6) may be used, but for highly stressed parts, rivets of an aluminum base alloy containing 4 to 5% magnesium are recommended. As a further precaution against electrolytic attack, the contacting surfaces of riveted magnesium alloy sheets or extruded shapes should be painted before assembly. A synthetic resin varnish pigmented with $ZnCrO_4$ is recommended for this purpose.

When it is necessary to rivet magnesium alloy parts to other metals, electrolytic insulation must be provided. To prevent the possibility of cracks or notches that later will produce fatigue failures, rivet holes should be drilled instead of punched, the rivet size carefully selected to avoid overstressing the rivet hole during driving, and marring of the magnesium alloy part with riveting hammer should be avoided.

Conclusion

The past fifteen years have witnessed the recognition of aluminum as an engineering material in an ever-widening field of applications. This is due primarily to the relatively light weight of aluminum compared to that of other common metals. However, advantage could not be taken of this characteristic until alloys of high strength were developed. Especially in recent years, marked progress has been made in this direction. This has been particularly true in those fields, such as construction and transportation, in which the cost of moving the structure is a major item of operating expense. Although aluminum is a relatively new engineering material, the fund of information that has been developed regarding its properties and behavior, and the great variety of forms and sizes in which it has become available commercially, places it on an equal rank, in these respects, with the far older metals.

This paper indicates broadly the type and scope of the facts and materials now available.

Part II of this paper entitled, "Magnesium", shows that although that metal has not yet progressed as far as aluminum, it is following a somewhat similar path. As is the case with aluminum, so, also, the outstanding important characteristic of magnesium is its relative lightness. Marked improvement has been made in the properties of its alloys. Methods of fabrication have been developed so that, to-day, magnesium alloys are available in a great variety of cast and wrought products of known characteristics.

Acknowledgment

The writers wish to acknowledge their indebtedness to the Marion Steam Shovel Company, Aluminum Company of America, The Dow Chemical Company, and the American Magnesium Corporation, for assistance in supplying basic data, illustrations, and other material useful in the preparation of the paper.

CORROSION IN RELATION TO ENGINEERING STRUCTURES

BY JAMES ASTON[78], ESQ.

SYNOPSIS

The corrosion problem may be considered from two angles, each encompassing quite an extensive field, and worthy in itself of the space normally allotted to a technical paper. The one phase, dealing with the fundamental mechanism of corrosion, is necessarily theoretical, and of secondary interest to the practising engineer. The other phase, pertaining to the relative utility of metals in engineering structures, is undoubtedly of major interest to a gathering of civil engineers. This latter subdivision is quite complex, involving factors of experience, of economics, of conjecture, of controversy, and of personal opinion.

It is the intention of the writer to take the middle ground, in the belief that some knowledge of the fundamental mechanism of corrosion is a valuable background for a more adequate balancing of the many variables which are presented by the multiplicity of service conditions, and to interlink this discussion of the fundamentals with an appraisal of materials and protective measures which have a bearing upon prolonging the life of structures by combating the ravages of corrosion.

The seriousness of corrosion loss may only be guessed at on the dollar basis. One finds estimates running into hundreds of millions of dollars and even into the billions, as the annual toll. The resulting tax is enormous, and the tangible depreciation of the cost of abandoning structures is exceeded in a monetary sense by the more intangible effects of designing structures heavier than the requirements of working stresses, substituting more costly metals for others that would meet design conditions on a more economical basis, modifying design and fabrication, and applying paints and other protective coatings—all occasioned by the allowances necessary to offset or to minimize the effects of corrosion.

MECHANISM OF CORROSION

The electro-chemical theory is most generally accepted in explaining the mechanism of corrosion. Aside from the metal involved, the essential factors are moisture and oxygen. Water serves as the medium of electro-chemical reaction; it must be in contact with the metal, and its activity will be accen-

[78] Metallurgist, Pittsburgh, Pa.

tuated or decreased by salts or gases in solution, temperature, etc., somewhat in proportion to their effect in increasing or decreasing the electrical conductivity of the liquid. Oxygen, dissolved in the water, plays a dual rôle: (1) It serves to oxidize the products of reaction to the familiar form (in the case of iron) of rust; and (2) it is a depolarizer, neutralizing the effect of hydrogen formed as a product of the customary reactions, thus enabling a continuity of electro-chemical activity which otherwise would be suppressed.

In the past, for iron in particular, undue emphasis was placed upon impurity or heterogeneity as an all-important factor in the inception and progression of corrosion. More recently, however, extraneous factors affecting the surface characteristics of the metal, and influencing the contact relationships with it of moisture and oxygen, have been recognized as playing an important rôle, at times, even to the extent that they overshadow certain of the internal characteristics of the metal. Rust may now be claimed to play a part in the progression of corrosion fully comparable to the other factors which formed the early basis of the electro-chemical theory. This later conception has served to amplify the theory, and to explain certain occurrences more rationally, notably pitting, which, at one time, were only vaguely understood.

All corrosion phenomena and effects revolve around the three essential factors: (*a*) The metal or metals concerned; (*b*) the moisture; and (*c*) the oxygen. The absence of any one of the factors will generally suppress the activity in its entirety, except in some special cases where a minor rate of progression may be observed. The rate, and more particularly the type, of corrosion resulting, will be markedly influenced by the relative balance in Factors (*a*), (*b*), and (*c*). Consequently, environment plays an important rôle in the problem of corrosion as related to structures.

Types of Service

Corrosion has most commonly been divided into three fields: Atmospheric, immersion, and soil. The metal may be considered common to all classes, the differentiation being primarily linked with the relationships between moisture and oxygen in the three types. Under atmospheric conditions there is a superabundance of oxygen. Moisture is the governing factor, with respect to its quantity, kind, and intermittent occurrence. Heavy precipitation naturally tends to increase corrosion. Contaminating gases of industrial atmospheres or the salt sprays of seaside conditions are aggravating influences. Of major significance is the interval between precipitations, and the opportunity thus afforded for the corrosion products to dry. Conditions favoring precipitation followed by relatively rapid drying will tend to spread the rust, rather than to concentrate the corrosion in pronounced pitting. Atmospheric corrosion is characterized by small, shallow pitting quite closely and uniformly distributed over the affected area. The result is a fairly uniform attack of the surface.

Immersion conditions represent an abundance of moisture and a varying, often deficient, supply of oxygen. The latter is a governing factor. True

immersion conditions may be defined as those in which the metal surface is wetted continually with little or no drying of the corrosion products. The nature of the water may aggravate or retard the corrosion. Most salts in solution in moderate quantities accelerate the effect; but under certain conditions there may be film precipitations on the surface of the metal which are markedly beneficial. Temperature is an important factor, with the result that, other conditions being equal, the most aggravated effects are experienced under the conditions prevailing in hot-water supply systems, in boiler-feed waters, and in steam-condensate returns. It is important to bear in mind that the degree of corrosion is not in proportion to the purity of the water. Relatively pure, oxygenated waters are more to be reckoned with than contaminated waters which are foul with organic matter. The blanket statement may be made that immersion service presents the most variable and complex conditions in the entire corrosion problem. Its results are manifested as a rule in scattered pitting of a relatively large and deep character, with much of the surface showing little or no corrosion. The result will depend in a secondary manner upon the water characteristics and oxygen supply, with the primary influence quite largely dependent upon how continuously the metal surface was kept wet.

Soil corrosion may be considered as occupying a position intermediate between atmospheric and immersion service. It offers a great variety of conditions, such as those in which leachy salts affect the composition of the water, and those in which moisture is retained or drained from the metal surface. Inherently, the soil may be considered as a sponge, acting in a secondary capacity in relation to the nature, amount, and frequency of the moisture supply and its accompanying oxygen. The degree, and particularly the type, of corrosion will vary with the foregoing combination of influences. The corrosion may be similar to the uniform type characteristic of atmospheric conditions; it often shows the aggravated pitting typical of immersion service; and, not infrequently, it will display peculiarities which are akin to those present in the case of acid waters.

Factors Involved in Alleviation

With the foregoing exposition of fundamentals and mechanism as a background, it will be valuable to consider what may be done toward remedial measures which will be effective against the deterioration of engineering structures. Obviously, attention may be focused upon the metals involved, or upon the environment—that is, upon the prevailing moisture and atmospheric conditions.

With respect to environment, artificial measures are limited to special cases. In underground structures, better soil may be substituted in the contact zone and, particularly, benefit will result from proper attention to drainage and the elimination of moisture retentivity in this contact zone. Where liquid conveyance is a condition, as in pipe lines, benefit may result from an adjustment of water characteristics. A notable example is that of Coolgardie, Australia, where internal corrosion was reduced markedly in a large pipe line by reducing the dissolved oxygen content of the water by evacuation.

In numerous instances, moderate applications of lime have proved effective. On the other hand, certain water treatments, which have for their objective the extreme softening of water for domestic and similar uses, have been harmful from the corrosion viewpoint because they removed from the water the natural film protection which results from the raw water. Hardening treatment for extremely soft waters, elimination of bicarbonate content, neutralization of acidity, and the introduction of film-forming constituents, such as sodium silicate, have improved the water characteristics in the limited fields where such applications are practicable. De-aeration may be, and is being, resorted to in boiler-feed waters, in hot-water supplies, and in hot-water heating systems. In general, the efforts are confined to above normal temperature conditions, under which the corrosion is most aggravated and which enable the remedy to be applied most effectively.

Unfortunately, in dealing with structures of direct concern to the Structural Division of the Society, artificial alteration of the environment is limited to a narrow range of special cases. From a practical standpoint, one must cope with the natural surroundings as they exist, and must make allowances in design for such deterioration as may be indicated by experience for the particular environment prevailing. In some cases it may be logical to minimize deterioration by selecting suitable material and by protective metal surfaces. In general the battle with corrosion must be fought through the structure itself, rather than through its environment.

A most important phase of the problem is the selection of the metal. From the standpoint of the designing engineer, this must be primarily because of its fitness as a load-sustaining material, and, in most instances, corrosion, although it cannot be ignored, becomes a secondary feature.

In discussing the fitness of metals from the viewpoint of corrosion, one enters the most difficult field. Situations vary, metals vary, and there is no blanket rule by which any metal may be claimed to be best adapted to all conditions. This phase of the discussion is difficult, also, because it assumes a severely controversial and competitive aspect, in which personal experience and bias, and commercial associations will necessarily be provocative of argument. It will be the purpose of the writer to present a summary of opinion and of the status of various metals that are of interest to the structural engineer.

The Metals of Interest

For purposes of discussion the metals of interest may be grouped into the non-ferrous and ferrous divisions; with a further subdivision of the latter into the iron group, ordinary steels, and alloy steels.

In the non-ferrous group, this discussion is limited to copper and its chief alloys, brass and bronze, and to aluminum and its alloys. It may be rightly contended that copper is selected for structural purposes because of its non-corrodibility or because it is ornamental. In no case is its choice an acknowledgement of superior, or even equal, physical properties of strength, ductility, or unit weight obtained at a competitive cost figure in comparison with structural steel.

For the aluminum group of metals, the same general statements may be made, except for the important modification that the desired physical properties may be obtained with lessened weight of structure, and this may dictate the choice of material. As a specific example, the use of aluminum floorbeams in the Smithfield Street Bridge, in Pittsburgh, Pa., may be justified from the engineering viewpoint as prolonging the useful life of an existing superstructure by lessening the dead load and thereby coping with the necessity of taking care of the heavier live load of existing traffic. In a somewhat similar manner, the higher cost of aluminum window frames and other parts of the Empire State Building, in New York, N. Y., may be justified or even offset, because of a lessened load on a costly supporting structure.

For the usual corrosion conditions prevailing in the realm of structural engineering, the metals of the foregoing non-ferrous group may be claimed to have a high order of immunity. Special conditions or locations may be cited as exceptions, without defeating the generalization. In all cases, their use in structures is of a limited, specialized nature. Occasionally, their corrosion value is a dominant factor in their selection, but more often this desirable characteristic may be taken as an added feature in a choice made because of special considerations related to design.

Except for very special conditions and applications along the lines noted, the structural engineer will confine his selection of a metal to the ferrous group. Economic considerations, necessitating individual analysis and judgment, will determine the choice among the several types of iron or steel comprising the group as a whole. The presentation which follows is an attempt at a general appraisal of their features of merit or demerit.

It is logical that ordinary steel should be considered first since it is, and probably always will be, the dominant metal for structures. Furthermore, it serves as a well understood standard against which the alternative metals may be properly appraised.

Structural Steel

The "Age of Steel" began with the inception of the Bessemer process in 1855, and the open-hearth method of Siemens shortly thereafter. Since then tremendous progress has been made in a technical as well as in a manufacturing sense. One may claim that the customary type of structural steel represents the "bed-rock" standard in its all-around features of cost and physical properties, to meet the general need. It is an adaptable material in the hands of the metallurgist, who finds that, through adjustment of chemical composition, aided by heat treatment where desired, a wide range of controlled properties is obtainable. The customary metalloids—carbon, silicon, sulfur, phosphorus, and manganese—may be added or held to any practicable limits, even to a virtual approach to zero. Limits are set by the cost of the operation in comparison with the advantages accruing.

For the foregoing five elements, and in the range usually encountered in ordinary steels, one may summarize the following facts concerning corrosion.

Carbon is added, as a rule, for the purpose of conferring the desired tensile, elastic, and ductile properties. Although there is an apparent gain, from

the corrosion viewpoint, as the carbon content is progressively lowered, the overwhelming advantage in fixing the percentage in accordance with the dictates of a standard fixed design relegates the corrosion factor into the background. Within the usual range, the influence of carbon upon the corrosion of the structure may be considered as relatively unimportant.

Silicon and manganese are the steel-makers' correctives, introduced to neutralize otherwise deleterious effects upon the physical properties. Although each may influence corrosion adversely as the quantity in the steel increases, the effect is no doubt small. In much of the literature and theory relating to corrosion, manganese has been given unwarranted prominence as a harmful agent. The structural engineer may rightly disregard these elements in the quantities customary in the steels that are of commercial interest, as serious factors in the corrosion of the structure.

Available evidence indicates that phosphorus has a retarding influence on corrosion, varying with conditions of service and probably being most pronounced in the case of atmospheric exposure. However, the well-known effect of phosphorus in promoting brittleness in steel will continue to dictate the low-phosphorus content of structural steel specifications. Design, and not corrosion, considerations will rule.

Sulfur is invariably recognized as being harmful from the corrosion viewpoint. It should be kept as low as possible with due recognition of the factors prevailing in any particular situation and of the added cost where abnormally low limits are demanded.

As a generalization, it may be stated that composition, within the specification limits set for plain structural steels, has little bearing upon its life from the corrosion angle. This generalization is subject to adverse modification, however, if steel is made so carelessly as to be characterized by segregation, non-uniformity, dirtiness, etc.

Cast and Wrought Iron

Before the introduction of steel, the iron group—namely, cast iron and wrought iron—formed the dominant materials available for structural uses. Cast iron has had an enviable record for corrosion service, especially for atmospheric and soil conditions. It is interesting to note that this is in spite of the fact that it is chemically the least pure and homogeneous of the ferrous metals. It is not reconcilable with a very widespread assumption that purity represents the goal in corrosion immunity; and it does lend support to the barrier principle of protection. In the case of cast iron, as corrosion proceeds, an increasing surface barrier of graphite flakes, aided perhaps by a tightly adherent layer of the corrosion products, serves to slow down, and to spread, the rusting effects. However, the brittleness and low tensile strength of cast iron, and its lack of desired fabricating qualities, will hold it to a very restricted and specialized place in modern engineering of structures.

One cannot impeach the integrity and experiences of those who believe that steel has failed to measure up, in corrosion service, to the standards set by wrought iron in the long history of its use. The Pillar of Delhi is a conspicuous example. England and Continental Europe contain many

examples of a later date, whereas wrought iron dating from the Colonial period of United States history, such as the Bowling Green fence in New York, N. Y., furnishes factual evidence to support the contentions of wrought-iron adherents. Furthermore, one cannot dismiss lightly the contemporary records within the relatively brief period which defines the Age of Steel.

Wrought iron has a place in contemporary engineering; that is, in specialized applications where corrosion and shock or fatigue stresses are encountered. Limitations in manufacture have been handicaps, resulting in restriction of markets to a few specialized lines, such as pipe and bar iron; and cost problems have narrowed the zone of utilization. The manufacturing handicaps for wrought iron have been overcome, and one should expect an expansion in its use, both in quantity and in diversification of application. In the structural field, aside from possible fatigue considerations, wrought iron offers no advantage over steel, from a stress or fabrication viewpoint. It must find its place where experience indicates that it will give an economic return in the necessarily specialized fields of utilization. Superior resistance to corrosion will be the basis of selection.

Commercially pure iron, or ingot iron, is a contemporary metal of importance. It is interesting to note that its development is due to an acceptance of the principle that wrought iron had rust-resisting qualities superior to steel and to a belief that high purity of base metal was the fundamental feature contributing to this superiority. This subject is the center of much commercial controversy. The steel interests contend that nothing has been gained over what may be expected with customary grades of mild open-hearth steel. The wrought-iron advocate will not admit that open-hearth iron has proved an adequate substitute for his products; and quite rightly contends that it is not wrought iron. Although the two have in common a high base metal purity, the ingot iron lacks the physical incorporation of slag filaments which are characteristic of wrought iron.

Alloy Steel

In considering the alloy steels, one is in the fairyland of ferrous metallurgy. The field is relatively unexplored and seemingly unlimited in extent. Actually, however, in the light of present knowledge and requirements, the useful territory is somewhat restricted. In comparison with customary steels, the alloy additions must result in improved strength characteristics, in comparable manufacturing and fabrication qualities, and must have a reasonable cost. To-day, the field seems to be covered by additions of chromium, copper, manganese, molybdenum, nickel, silicon, and vanadium—together with carbon—singly, or in various combinations. Covering the low-alloy steels Mr. J. C. Whetzel[74] gives the general range of elements, as follows:

Carbon	0.10 to 0.40	Copper	0.01 to 1.40
Manganese	0.20 to 1.70	Nickel	0 to 3.5
Phosphorus	0.01 to 0.20	Chromium	0 to 12.0
Sulfur	0.05 maximum	Molybdenum	0 to 0.40
Silicon	0 to 1.0	Vanadium	0 to 0.20

[74] *Proceedings*, Am. Iron and Steel Inst., May, 1935.

From the corrosion viewpoint, it appears that the carbon-silicon-manganese group has no particular merit and that the results of promise are in the combinations containing copper, nickel, and chromium.

Nickel steel (3.5% nickel) has been a standard, high-strength structural steel for many years. It has had a reliable performance record with high-strength characteristics, without the necessity of special heat treatment and radical deviations in manufacture and fabrication. The cost of the nickel addition has been the factor that has held this material to specialized applications in structures. The nickel content of the steel no doubt confers added corrosion resistance, but scarcely in proportion to the additional cost. Consequently, the use of nickel steel would not appear to be justified from the corrosion standpoint alone.

One of the most successful developments has been copper-bearing steel which has been successful because a small quantity of copper (0.30%) confers material benefit at low cost. Comprehensive tests by the American Society for Testing Materials indicate a substantial corrosion resistance under atmospheric conditions, particularly in industrial centers. A similar merit, however, cannot be claimed for immersion conditions. The added cost of this copper-bearing steel must justify its selection solely because of its corrosion characteristics; no benefit may be claimed because of its other physical properties in comparison with standard structural steels.

The results with higher copper additions (0.50% to 1.5%) are rather doubtful as far as structural applications are concerned. There is higher strength, and probably greater corrosion resistance, but the relationships of copper and iron in these higher ranges may lead to difficulties in manufacture and to unreliability in fabrication and in after service. It is not improbable that copper will play an important rôle in subsequent developments by recourse to other auxiliary alloying elements which will serve to shift the normal relationships of copper and iron, or to mask the otherwise detrimental effects. It may be that molybdenum has an influence of this character.

Low Alloy Structural Steel

In the past few years there has been a marked impetus in the development and exploitation of new steels in the low-alloy group. Numbers of these alloys are on the market under commercial trade designations. The general types are well summarized by Whetzel[74] who gives the chemical composition, tensile and impact properties, and other characteristics. With a demand for lighter structures, the steel industry has endeavored to meet the competition of low-density metals by offering higher strength characteristics and consequent lessened section and weight. Quite naturally the greatest impetus has come through the transition which one observes in the transportation field. Promoters of the new low-alloy steels have the objectives of stepping into the territory so ably filled in the past by the use of nickel steel, but at a reduced cost. Manganese, silicon, and copper are the agencies presenting the greatest possibilities, because of the moderate required amounts and the low base cost of the alloy metals. Alone, or in various combinations, and often in association with

nickel, chromium, molybdenum, and vanadium, one finds, to-day, the general list confronting the structural engineer.

As a matter of fact, the manganese and silicon structural steels have been used for a considerable period. From the corrosion viewpoint, they may be likened to ordinary steel and their selection should be made only because of benefit from a design basis.

In appraising the low-alloy steels from the standpoint of corrosion service, one quite commonly overlooked feature is worthy of emphasis. The basis of selection is almost invariably higher unit strength and consequent reduction of weight. In most structural shapes, this means little change in the over-all surface exposed to the elements and a reduction of section thickness in proportion to the increased strength of the metal. Consequently, if the strength is doubled, and the section thickness is reduced one-half, with a unit corrosion-merit value of twice that of ordinary steel, the life of the structure, as far as comparative corrosion is concerned, would be the same. In effect, therefore, in the selection of structural materials, the benefit to be derived, as far as it relates to corrosion, will be only to the extent that the corrosion-merit ratio exceeds the strength ratio for the metals under consideration.

In general, a proportionately high order of corrosion resistance in the low-alloy group does not appear to be attained unless the quantities of chromium or nickel, singly or in combination, are relatively high. In such a case the engineer is faced with a high-cost spread and the possibility of manufacturing and fabricating peculiarities. The writer believes that selection of metals in the low-alloy group would not be justified in engineering structures, as a major substitution for plain steels, if all other physical factors were the same, and corrosion was the only service consideration. On the contrary, where design characteristics of the several types dominate the selection, the equivalent corrosion characteristics may be considered as an additional premium obtained with certain of the materials; or the selection may be made by proper appraisal of all factors—strength, corrosion, and cost—peculiar to the competing metals. Obviously, with this criterion as the logical basis of choice, each structure becomes an entity unto itself—a problem for individual solution.

As illustrative of some considerations involved in any generalization regarding the low-alloy steels, one may cite an interesting alloy combination noted by Whetzel[74]. A chrome-copper-silicon-phosphorus addition gives to this steel the properties of the medium high-tensile group. It is claimed that the detrimental effects to be expected from abnormally high phosphorus (0.10% to 0.20%) are masked by the complex alloying relationships, and cold shortness is not induced in the metal. Graphs are given by Whetzel showing comparative corrosion in industrial atmosphere for two years, and the citation is made that copper-bearing steel has a resistance of two to three times that of ordinary steel, whereas for the chromium-copper-silicon-phosphorus combination the ratio is four to six times. If these comparisons are borne out by long-time service experiences, one would have a rational basis for the evaluation of the alloy for structural needs, with adequate balancing of all factors—strength, corrosion, and cost. Such evaluation at

present would be unwise, since ultimate life cannot safely be predicted upon short-term, progress data. Furthermore, in the corrosion field, results vary markedly with conditions of service.

The greatest promise in the structural field is in adding alloys of low cost, in quantity and kind, which will give a positive corrosion merit warranting selection in competition with the usual grade of structural steel even if there should be little or no gain in the other physical properties.

One might almost claim that corrosion, for the conditions confronting the structural engineer, has been conquered by alloys of the 13.0% to 16.0% chromium, or the 18.0% chromium with 8.0% nickel classes—the high-alloy group. These steels have the ideal quality of forming thin self-healing surface films in usual environments which, in the last analysis, is the goal that the corrosion specialist dreams of achieving. However, aside from questions of fabrication and peculiarities of properties under certain conditions, stainless steels have very limited, specialized applications in structures because of a relatively high unit cost.

Surface Protection

Corrosion is a surface reaction, so that surface considerations or applications form a logical method of attacking the problem. Furthermore, in view of the fact that a ferrous metal of moderate cost is, and will remain, the dominant one for structures, and that rusting will occur in some degree, paint or equivalent surface applications are required in most installations, if only for æsthetic reasons. An admirable paper on the surface protection of steel has been presented by Mr. F. N. Speller[75]

Of the metallic coatings, interest centers upon hot-dip applications of zinc or tin. Of the two, galvanizing only need be given serious attention because of the high cost of tin. Although galvanizing forms a good protection for most exposure conditions, the cost is relatively high. Painting is usually a necessary accompaniment; and application of zinc is impracticable on the sizes and shapes used in large structures. In certain cases, such, for example, as bridge parts to which access for painting after erection is difficult, a zinc coating applied by hot-dipping, or by the metal-spray method, may be advantageous. Every one is familiar with the extensive use of galvanizing in building sheathing and in electrical transmission towers. In general, however, zinc and metallic coatings, as a whole, are of minor interest to the structural engineer.

As a protection against corrosion, the non-metallic coatings have merit only because they isolate the metal surface from contact with its environment —moisture in particular. The necessary qualifications are firm adherence to the underlying metal, a continued imperviousness, and resistance to disintegration during exposure. The metal surface is an important feature since dampness, grease, or rust affect adherence. Furthermore, the evidence is that iron or steel which corrodes to a dense, adherent rust is of decided advantage in the effectiveness of a paint coating, in contrast with results where the steel corrodes to a flocculent, loose type of rust. Paints containing pigments of the red lead, or of the chromate, types appear to have an inhibitive action on

[75] *Transactions*, A. S. M. E., June, 1935.

the corrosion of the steel base. To this extent they are of advantage in the contact zone; that is, in the priming coat.

Paints comprise the most important material available for the surface protection of structures. Coatings of greater thickness, such as bituminous or Portland cement mixtures, are more commonly applied in underground protection. In such cases, inaccessibility after erection demands a coating of reasonable permanence, since repeated renewal, as in painting, is practically out of the question. The fundamental features of protection are similar to those noted previously. In addition, the bituminous coatings should possess sufficient plasticity to yield under slight movement, and to resist cracking, and yet be of proper consistency to hold up under temperature influences and to resist deformation under earth pressures. For pipe lines, as an example, this has been satisfactorily accomplished by reinforcement with wrappings of burlap or asbestos embedded in the bitumen.

The Future

The metallurgist has advanced materially in his knowledge of the mechanism of corrosion. Likewise he has made progress in the improvement and development of materials for utilization by the engineer against its ravages. For this latter, more practical, accomplishment one may credit the eternal rivalry and struggle for supremacy among the several competing groups involved in the manufacture of materials for structural uses. One may look into the future with confidence of further progress which will be measured by: (*a*) Betterment of existing materials; (*b*) possible development of new types; and (*c*) reduction of cost spreads which will re-orient the economic basis of selection. After all, corrosion is an economic problem to an even greater degree than it is a technical one. There is no specific formula by which metals may be evaluated; results under one set of conditions are not necessarily transferable to another environment. Unfortunately, there is yet, no quick time test for appraising comparable to the tensile test which makes the diagnosis of physical properties reasonably satisfactory. Experience and good judgment remain as the best guide in selecting materials for corrosion service.

On the side of the structural engineer, the advances in knowledge and in materials open up a broader field for selection and economic placement. There are numerous places where special materials, even at higher cost, are justified from the viewpoint of corrosion alone. In bridges, there are inaccessable parts, such as connections and floor members; and, in buildings, harmful conditions may be encountered in foundations and where there is exposure to the atmosphere. Submerged structures (piers, etc.) present a serious problem, whereas the atmospheric conditions associated with marine and many industrial structures are conducive to accelerated deterioration. For these special cases it is good engineering to weigh all factors carefully and to select such metals or protective measures as experience and good judgment may indicate. However, in the broad field of structural engineering, and in spite of prospective development in the higher strength alloy steels, plain carbon steel will continue as the dominant material. The corrosion problem will remain as a serious handicap. The best defense will be "keep on painting."

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

STRUCTURAL APPLICATIONS OF STEEL AND LIGHT-WEIGHT ALLOYS

A SYMPOSIUM

PAGE

III.—STRUCTURAL APPLICATIONS OF SPECIAL STEELS:

Actual Applications of Special Structural Steels.
By V. D. BEARD, M. AM. SOC. C. E. 1252

Evolution of High-Strength Steels Used in Structural Engineering.
By LEON S. MOISSEIFF, M. AM. SOC. C. E. 1277

ACTUAL APPLICATIONS OF SPECIAL STRUCTURAL STEELS

By V. D. Beard[76], M. Am. Soc. C. E.

Synopsis

Within the meaning of the title selected for this paper it seems advisable first to preface the discussion by mentioning the steel adopted as the standard of reference, and then to give a list of materials to illustrate what are properly designated "Special Structural Steels":

(*a*) The standard selected as most suitable for the purpose is Carbon Steel A. S. T. M. Specifications A7 and A9. Comparisons of the special steels will be referred to this standard.

(*b*) Special structural steels, which draw their distinction from their physical properties, or their resistance to corrosion, or in some cases from both, afford a considerable variety. Representative members of this group are the following: High-carbon heat-treated steels; carbon steels containing copper; silicon steel; manganese steel; nickel steel; manganese vanadium steel; Cor-Ten; Cromansil; chromium steels; the stainless steels; and carbon steel, cold-drawn, for cables and wire rope.[77]

Bridges — Long Spans and Monumental Structures

Typical examples of structures in which special steel was used, are as follows.

San Francisco-Oakland Bay Bridge.—This great structure symbolizes an epoch in American bridge building. Approximately 4.5 miles of the 8.5-mile crossing consists of various types of bridge construction in which 200 000 tons

TABLE 11.—Physical Characteristics of Special Structural Steels

Description (1)	Unit Stresses, in Kips per Square Inch: Ultimate strength (2)	Yield point (3)	Working stress (4)
Medium carbon steel	62	37	28
Silicon steel	80	45	28
Nickel steel	85	55	34
Heat-treated eye-bars	80	50	34
Cable wire	220	150	82

of steel were used. The individual units, when viewed apart from the entire project, are notable bridges in their own right, and merit rather detailed description because of the extensive use of special structural steels. These

[76] Designing Engr., Am. Bridge Co., Pittsburgh, Pa.

[77] Progress Rept., Sub-Committee No. 2, Committee on Steel of the Structural Division, Am. Soc. C. E., on Structural Alloy and Heat-Treated Steels, *Proceedings*, Am. Soc. C. E., March, 1936, p. 361.

steels are classified as shown in Table 11 with regard to ultimate strength, yield point, and unit stresses.

The West Bay Crossing, from San Francisco, Calif., to Yerba Buena Island, is composed of two, nearly duplicate, double-deck, parallel wire cable, sus-

TABLE 12.—Basic Data, Long-Span Bridges

Description	George Washington Bridge	San Francisco–Oakland Bay Bridge: West Bay Crossing	San Francisco–Oakland Bay Bridge: East Bay Crossing	Golden Gate Bridge	Ambassador Bridge	Delaware River Bridge	Bayonne Bridge
Type of structure	Suspension	Suspension	Cantilever	Suspension	Suspension	Suspension	Steel arch
Length of main span, in feet	3 500	2 310‡	1 400	4 200	1 850	1 750	1 675
Length of side spans, in feet	......	1 155	500	1 125	......	752	....
Weight of structural steel, in tons	......	84 700§	59 000	......	......	......	17 160
Cables:							
Diameter, in inches	36*	28¾	......	36.25‡	......	30	....
Number of galvanized wires	26 474	17 464	......	27 572	......	......	....
Diameter of each wire, in inches	{ 0.196	...... 0.195	......	0.192	......	...... 0.196	....
Maximum stress, in kips per square inch	82	82	......	82	......	72	....
Temper	Cold drawn	Cold drawn	......	Cold drawn	Cold drawn	Cold drawn	....
Ultimate strength, in kips per square inch	220†	......	......	220	215	220	....
Yield point stress, in kips per square inch	......	......	......	160	144	144	....

* Four each. † Average must be 225 kips per sq. in. ‡ Two each. § Total weight of two bridges.

pension bridges (see Table 12). The following tabulation will give an idea of the various types of steel used and their relative weights:

Distribution	West Bay Crossing, San Francisco-Oakland Bay Bridge (Tons)	Distribution	West Bay Crossing, San Francisco-Oakland Bay Bridge (Tons)
Silicon steel in towers, cable bents, and center anchorage	18 500	Phosphor bronze	25
Heat-treated eye-bars and pins	350	Cable wire	18 700
Nickel steel	280	Cable castings	830
Silicon steel on suspended structure.....	24 000	2.25-in. suspender ropes.	113
Manganese steel rivets..	300		

The percentage of this weight that was of special steel may be noted by reference to Table 13(*a*). The East Bay Crossing, from Yerba Buena Island to Oakland, Calif., is composed of four 280-ft deck spans, one cantilever bridge with a 1 400-ft main span, and two 500-ft side spans, five 504-ft through spans on bents, fourteen 280-ft deck spans on steel bents and masonry piers, and ten 82-ft plate-girder spans. A comparison of this 1 400-ft cantilever with the Quebec and Firth of Forth Bridges is shown in Table 14. The special steels in this structure, their distribution in the various parts of the structure, and the percentage of the total tonnage, are given in Table 13(*b*).

TABLE 13.—Distribution of Special Types of Steel in the San Francisco-Oakland Bay Bridge (Weight, in Tons)

Type	(a) West Bay Crossing					(b) East Bay Crossing					
	Towers 2, 3, 5, and 6	Pier No. 4	Stiffening trusses	Floor-beams and stringers	Total	Cantilever span	Towers and bents	500-foot span	Lattice truss spans	Girder spans	Total
Silicon	17 500	500	15 000	6 950	39 950	7 400	3 525	6 500	10 250	350	28 025
Nickel	......	280	......	.....	280	3 400	.....	.....	......	.....	3 400
Manganese rivets	......	.....	300	.....	300	25	.....	.....	......	.....	25
Phosphor bronze	......	.....	24	.....	24	.....	.....	.....	......	.....	....
Heat-treated steel	......	.....	......	.....	......	3 050	.....	1 200	......	.....	4 250
Percentage of total weight	79	62	82	52	......	77	64	62	52	43	64

Tables 15(*a*) and 15(*b*) show the chemical and physical properties of the special steels: For nickel (Table 15(*a*)) as averaged from the first ten melts; and for silicon (Table 15(*b*)), as averaged from thirty melts, ten from each of three mills. There are 370 tons of low-nickel chrome heat-treated

TABLE 14.—Comparison of Material Used in These Cantilever Bridges

Bridge	Span length, in feet	Total weight, in tons	Material Used (Percentages)			
			Nickel	Silicon	Heat-treated bars	Carbon
San Francisco–Oakland Bay	1 400	20 900	16	40	10	34
Firth of Forth	1 700	30 000	....	....	....	100*
Quebec	1 800	62 900	27	....	....	73

* Medium carbon steel.

pins in the cantilever structure which vary in size from 11½ in. to 24 in. in diameter and which have the chemical and physical properties shown in Table 15(*c*). The manganese rivets were furnished to the specifications shown in Table 15(*d*).

Golden Gate Bridge.—The Golden Gate Bridge is the longest single suspension bridge in the world (see Table 12). It is a single-deck structure and is 90 ft wide from center to center of stiffening trusses, providing for a 60-ft roadway and two 11-ft sidewalks. The 700-ft towers are of the fixed-base, flexible type. The live load on the stringers was four 24-ton trucks and two 50-ton street cars abreast, with 50% impact for stringers and 25% for floor-beams.

Silicon steel was used for the top 200 ft of the towers and for the cross-strut below the roadway. The allowable unit stresses were 24 kips per sq in. on silicon steel and 18 kips per sq in. on carbon steel, except as follows (units are kips per square inch):

Carbon steel in stiffening trusses	21
Silicon steel in stiffening trusses	32
Silicon steel in lateral bracing	32
Silicon steel in flanges of floor-beams	22
Carbon steel in webs of floor-beams	16

There were 20 000 tons of silicon steel used on this bridge, and 2 500 tons of heat-treated eye-bars were used in the anchorages.

TABLE 15.—CHEMICAL COMPOSITION AND PHYSICAL CHARACTERISTICS OF SPECIAL STRUCTURAL STEELS

Item No.	Remarks	Chemical Composition (Percentages): Carbon	Manganese	Phosphorus	Sulfur	Nickel	Silicon	Copper	Chromium	Unit Stresses, in Kips per Square Inch: Yield point	Ultimate strength	Percentage elongation in 2 in.	Percentage reduction in area
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
	(a) NICKEL STEEL IN THE SAN FRANCISCO – OAKLAND BAY BRIDGE												
1	Average	0.29	0.62	0.015	0.023	3.40	0.21	0.28	.....	65.5	97.4	18.6	38.6
2	Minimum	0.25	0.50	.....	.....	3.25	0.16	0.26	.....	59.5	90.6	16.2	30.5
3	Maximum	0.35	0.72	.....	.....	3.69	0.25	0.35	.....	72.6	104.6	21.2	47.3
4	Specified	0.40[p]	....	0.04[p]	0.05[p]	3.00[q]	.....	0.20[q]	.....	55.0[q]	90.0[q]	12[q]	30
	(b) SILICON STEEL IN THE SAN FRANCISCO – OAKLAND BAY BRIDGE												
5	Average	0.31	0.82	0.02	0.03	....	0.273	0.265	.....	53.9	89.9	22.8	46
6	Minimum	0.26	0.68	.....	.....	....	0.21	0.20	.....	46.4	81.0	19.	34.8
7	Maximum	0.35	0.95	.....	.....	....	0.34	0.33	.....	63.9	92.5	26.2	55.8
8	Specified	0.40[p]	1.00[p]	0.04[p]	0.05[p]	....	0.45[r]	0.20[q]	.....	45.0[q]	95.0[s]	14	35
	(c) LOW NICKEL, CHROMIUM, HEAT-TREATED PINS; EAST BAY CROSSING, SAN FRANCISCO – OAKLAND BAY BRIDGE												
9	Average, twelve heats	0.33	0.64	0.026	0.025	1.36	0.20	....	0.65	64.2	97.3	21	53.7
	(d) MANGANESE RIVETS IN THE SAN FRANCISCO – OAKLAND BAY BRIDGE												
10	Specified	0.30[p]	1.35[p]	0.04[p]	0.05[p]	....	0.25[p]	0.20[q]	.....	42	90[t]	....	45
	(e) SPECIFICATIONS FOR SPECIAL MANGANESE STEEL, BAYONNE (N. J.) BRIDGE												
11	Structural steel	0.40[p]	1.80[p]	.....	.....	....	0.30[u]	....	.....	55	90	....	....
12	Rivets	0.35[p]	1.80[p]	.....	.....	....	0.30[u]	....	.....	47	100[v]	....	....
	(f) HEAT-TREATED EYE-BARS IN THE HUEY P. LONG BRIDGE, AT NEW ORLEANS, LA.												
13	Average	0.35	0.63	.....	.....	....	0.08	....	.....	58.5	89.0	10[w]	....
14	Minimum	0.31	0.53	.....	.....	....	0.060	....	.....	52.7	82.1	7.6[w]	....
15	Maximum	0.39	0.71	.....	.....	....	0.112	....	.....	64.4	97.4	14.7[w]	....
16	Specified	0.30[q]	0.60[q]	.....	.....	....	.....	....	.....	50.0	80.0	8.0[w]	....
	(g) SPECIAL HEAT-TREATED EYE-BARS IN THE POINT PLEASANT BRIDGE (W. VA.)												
17	Average	....	....	.....	.....	....	.....	....	.....	81.0	114.5	6.8[w]	....
18	Minimum	....	....	.....	.....	....	.....	....	.....	78.3	111.6	4.1[w]	....
19	Maximum	....	....	.....	.....	....	.....	....	.....	85.7	119.6	7.7[w]	....
20	Specified	....	....	.....	.....	....	.....	....	.....	75.0	105.0	5.0[w]	....
	(h) TESTS OF STRUCTURAL STEEL FROM THE RICHMOND POWER STATION, PHILADELPHIA, PA.												
21	Minimum	0.31	0.68	.....	.....	....	0.15	....	.....	41.5	80.2	18.2	....
22	Maximum	0.39	0.90	.....	.....	....	0.33	....	.....	53.1	94.1	24.5	....
	(i) STANDARD MANGANESE BOLTS												
23	Specified	0.30[p]	1.35[q]	0.04	0.05	....	.....	....	.....	42	90[t]	[x]	....
	(j) GRILLS FOR THE KANAWHA POWER PLANT, HAWKS NEST, W. VA.												
	Range:												
24	From	0.15	1.10	.....	.....	....	0.70	....	0.40	60[q]	100[q]	18	....
25	To	0.35	1.50	.....	.....	....	1.00	....	0.70				
	(k) TOWERS OF THE PHILADELPHIA – CAMDEN BRIDGE												
26	Average	0.35	0.76	.....	.....	....	0.27	....	.....	....	....	....	....
	(l) TOWERS OF THE GOLDEN GATE BRIDGE												
27	Average	0.27	1.20	.....	.....	....	0.24	....	.....	....	....	....	....

[p] Maximum allowable. [q] Minimum allowable. [r] Minimum allowable value, 0.20 per cent. [s] 80 to 95 kips per sq in. [t] 75 to 90 kips per sq in. [u] From 0.10 to 0.30 per cent. [v] From 82 to 100 kips per sq in. [w] Percentage elongation in 18 ft. [x] For all items in this section, the percentage elongation in 8 in. $= \frac{1500}{\text{Ultimate strength}}$.

TABLE 15.—(*Continued*)

Item No.	Remarks (1)	Chemical Composition (Percentages): Carbon (2)	Manganese (3)	Phosphorus (4)	Sulfur (5)	Nickel (6)	Silicon (7)	Copper (8)	Chromium (9)	Unit Stresses in Kips per Square Inch: Yield point (10)	Ultimate strength (11)	Percentage elongation in 8 in. (12)	Percentage reduction in area (13)
(m) Tests to Observe Difficulties of Fabricating High-Tensile Steel with Present Shop Equipment (See Table 17)													
	Brinell Hardness Nos.:												
28	228	0.45	2.00	.....	.....	....	.....	....	.....	82	126	9.5	7.6
29	228	0.32	1.95	.....	.....	....	.....	....	.....	75	109	16.0	42.4
30	228	0.32	1.95	.....	.....	....	.....	....	.....	75	109	16.0	42.4
31	207	0.33	1.54	.....	.....	....	0.25	....	.....	61	100	16.7	46.7
32	207	0.34	1.57	.....	.....	....	2.24	....	.....	65	102	19.5	53.6
33	207	0.34	1.57	.....	.....	....	0.24	....	.....	65	102	19.5	53.6
34	217	0.36	1.93	.....	.....	....	0.25	....	.....	78	102	1.5	2.5
35	228	0.36	1.90	.....	.....	....	0.28	....	.....	80	127	11.5	17.9
36	207	0.29	1.08	.....	.....	2.12	0.30	....	.....	64	100	16.0	51.8
37	183	0.29	1.08	.....	.....	2.12	0.30	....	.....	64	100	16.0	51.8
38	207	0.28	1.01	.....	.....	2.12	0.32	....	.....	65	100	19.5	42.5
39	217	0.35	1.85	.....	.....	....	0.28	....	.....	70	112	12.0	25.0
40	228	0.35	1.85	.....	.....	....	0.28	....	.....	70	112	12.0	25.0
41	217	0.29	1.09	.....	.....	2.01	0.26	....	.....	67	106	17.7	37.4
42	212	0.29	1.09	.....	.....	2.01	0.26	....	.....	67	106	17.7	37.4
43	196	0.30	0.83	.....	.....	2.10	0.26	....	.....	61	102	18.2	42.7
44	207	0.30	0.83	.....	.....	2.10	0.26	....	.....	61	102	18.2	42.7
45	207	0.28	0.97	.....	.....	2.13	0.26	....	.....	60	99	18.0	49.5
46	207	0.30	1.10	.....	.....	2.05	0.25	....	.....	66	106	17.5	43.2
47	228	0.35	1.89	.....	.....	....	0.29	....	.....	70	110	15.0	44.3
48	228	0.35	1.89	.....	.....	....	0.29	....	.....	70	110	15.0	44.3
49	228	0.35	1.89	.....	.....	....	0.29	....	.....	70	110	15.0	44.3
50	241	0.35	1.89	.....	.....	....	0.29	....	.....	70	110	15.0	44.3
51	241	0.33	1.73	.....	.....	....	0.24	....	.....	58	102	15.0	20.9
(n) U. S. Navy Specifications for Special Steel Suitable for Welding													
52	No. 48 S 5 e	0.18	1.45	.....	.....	....	0.25	....	Va. 008/018	50	80	....	....
(o) Tests on Silicon and Nickel Steel for the Delaware River Bridge (See Figs. 34 and 35)													
53	Sheared plates, all 5/8 in. thick	0.32	0.65	0.014	0.033	....	0.300	....	....	42.7 47.0 47.2 45.9 46.5 47.0	74.5 81.8 82.7 82.3 83.0 82.3	26.7 23.2 25.5 23.7 23.5 25.0	42.7 40.0 43.3 41.8 45.2 47.4
54	Angles: 8 by 8 by 3/4 8 by 8 by 1 8 by 8 by 1	0.35	0.75	0.023	0.040	....	0.208	....	....	46.7 48.4 45.0	81.4 82.5 79.3	22.2 22.5 25.0	40.1 33.1 38.3
55	Sheared plates: 5/8 in. thick	0.38	0.60	0.025	0.036	....	0.216	....	....	49.0 48.4	82.3 82.9	25.7 24.5	40.8 44.4
56	Universal mill plates: 5/8 in. thick	0.35	0.86	0.025	0.040	....	0.280	....	....	47.7 50.8	89.0 88.4	24.2 21.7	50.1 47.4
57	Universal mill plate: 5/8 in.	0.34	0.70	0.024	0.041	....	0.208	....	.....	48.1	82.8	20.7	44.5
58	8 by 6 by 3/4-in. angles	0.39	0.72	0.016	0.026	3.28	0.052	....	....	72.8 80.2 75.2 80.1	121.0 119.0 121.0 121.2	13.7 8.2 14.7 12.7	30.1 29.2 33.2 43.0
59	Sheared plates: 13/16 in. thick 1/2 in. thick	0.31	0.57	0.013	0.027	3.23	0.080	....	....	59.5 56.9	101.0 92.0	20.5 21.5	43.5 50.4
60	Universal mill plates: 13/16 in. thick 7/16 in. thick	0.28	0.50	0.016	0.026	3.18	0.076	....	....	55.8 56.6	90.7 92.8	21.0 18.0	42.1 45.9
61	Sheared and Universal mill plates: 1 in. thick 3/4 in. thick 1 in. thick	0.30	0.55	0.016	0.034	3.28	0.068	....	....	56.0 55.6 55.6	95.3 95.2 97.4	21 7 20.5 18.2	39.7 41.3 39.8
62	Sheared plates: 13/16 in. thick 1/2 in. thick	0.28	0.51	0.013	0.033	3.28	0.060	....	....	56.3 54.1	89.2 86.0	20.7 22.0	43.2 45.5
63	....	....	....	.....	.....	....	.....	....	.....	....	....	....	....
64	8 by 8 by 5/8-in. angles	0.34	0.41	0.015	0.026	3.16	0.052	....	....	54.2 56.5	89.1 89.5	22.7 21.7	47.6 41.4

George Washington Bridge.—This well-known bridge over the Hudson River, in New York, N. Y., is a suspension bridge with a 3 500-ft center span. It is 106 ft wide, center to center of stiffening trusses[78]. Silicon steel was used for the chords of the stiffening trusses for the side spans, whereas nickel steel was used in the center span. Silicon steel was also used for the main columns of the towers and main material of the floor system, whereas carbon steel was used for the tower bracing and for details of the tower and floor.

The weight of steel on the bridge was divided as follows, in tons: Wire, 29 000; carbon steel, 31 000; silicon steel, 30 000; and nickel steel, 2 350. The towers contained 23 800 tons of silicon and 15 200 tons of carbon steel.

Ambassador Bridge.—This international bridge over the Detroit River at Detroit, Mich., is of the suspension type. The towers are 363 ft high. The 47-ft roadway carries five lanes of traffic, and there are two 8-ft sidewalks. The chords of the stiffening trusses are of silicon steel whereas the webs are of carbon steel for which unit stresses of 32 and 24 kips per sq in. were used. The cables were originally spun with heat-treated wire that had an ultimate strength of 220 kips per sq in. and a yield of 194 kips per sq in. This wire proved to be unsatisfactory when placed in the bridge, because of its inability to stand the bending stresses, and was replaced. The new wires were cold drawn (see Table 12), with the physical requirements of ultimate strength of 215 kips per sq in. and yield point of 144 kips per sq in., measured at an elongation of 75% in an original length of 10 in. A very extensive series of tests were made. Only 4 in 300 showed less than 215 kips per sq in., the lowest being 210 kips per sq in. A total of 45 yield-point tests met the specification requirements. Tension tests for permanent elongation indicated a proportional limit of 103 kips per sq in., which is well above design stresses. Creep tests showed very little difference between straight wires and those bent around sheaves. No wires broke over sheaves or at the point of tangency.

Delaware River Bridge.—This suspension bridge at Philadelphia, Pa., has a main span of 1 750 ft and side spans of 751 ft 8 in. The towers are 337.5 ft high. Three grades of steel were used in this bridge: Silicon steel for the main compression material of tower posts, web members, and lateral systems of stiffening trusses, and some of the heavy girders in the approaches; nickel steel for chords of stiffening trusses; and structural steel for the remainder. The two special steels were used to effect economy in weight (webs of stiffening trusses), or to secure great flexibility within the elastic limit of the material (in the towers, and still more so in the truss chords).

A unit stress of 24 kips per sq in. was used for the silicon steel in the towers for the sum of axial and bending stresses due to dead load, congested load, wind, and temperature. Including the secondary stresses, the allowable unit stress was 27 kips per sq in.

For the nickel steel (yield point, 55 kips per sq in.) in the chords of the stiffening trusses, a unit stress of 40 kips per sq in. in tension was used, and 35 kips per sq in. in compression, with the top chord working stress reduced 3 kips per sq in. on account of lateral forces; 32 kips per sq in. in

[78] *Transactions*, Am. Soc. C. E., Vol. 97 (1933).

tension was allowed for silicon steel in the web members and $32\,000 - 140 \frac{L}{k}$ lb per sq in. in compression. For the laterals, the allowable unit stress for silicon in tension was 32 kips per sq in. and $30\,000 - 100 \frac{L}{k}$ lb per sq in. in compression, but some value greater than 10 000 lb per sq in., of this was absorbed by participation in the chord stresses.

The Bayonne Bridge.—This bridge, which spans the Kill van Kull, is the longest arch span in the world. It has a span of 1 675 ft, a rise of 274 ft, and a width of 74 ft, center to center of trusses.

The total weight of steel in the arch is 17 160 tons, of which 4 000 tons is silicon and 4 700 tons is manganese steel. The 2 000 ft of viaduct approaches contained 4 150 tons of silicon and 350 tons of manganese steel. The allowable unit stresses on the three materials, in kips per square inch, were: For carbon, 20; for silicon, 27; and for manganese steel, 33. The special manganese steel for this bridge was specified as shown in Table 15(*e*).

Bridges—Movable Spans

The Cape Cod Canal Railroad Lift Span, at Buzzards Bay, Massachusetts, the longest lift span on record, is 544 ft, center to center of piers. The lift span is composed of silicon steel, except the chords in the end panels (see Table 16).

TABLE 16.—Basic Data—Movable Bridges of Medium Length

Description (1)	Buzzard's Bay Railroad Bridge across Cape Cod Canal (2)	Burlington to Bristol Bridge, across the Delaware River (3)	Railway bridge, at Boonville, Mo. (4)	Hackensack River crossing (5)	West Madison Street Bridge, Chicago, Ill. (6)	North Wabash Avenue Bridge, Chicago, Ill. (7)	South Harlem Avenue Bridge, Chicago, Ill. (8)	Outer Drive Bridge, Chicago, Ill. (9)
Type	Vertical lift	Vertical lift	Vertical lift	Vertical lift	Double-leaf bascule	Double-leaf bascule	Double-leaf deck trusses	Double-leaf deck trusses
Span length, in feet*	544	533.75	408.0	198	221.23‡	269	224	264
Weight of Steel, in Tons:								
Silicon	2 248	......	2 100	......	......	......	812	1 965
Carbon	2 144	......	......	......	......	......	......	......
Nickel	......	......	......	......	519	329	......	......
Width, in Feet:								
Center to center of trusses	27	......	20	45	......	......	......	......
Roadway	......	20	......	3†	38	60	56	38§

* Center to center of piers. † Tracks. ‡ Center to center of trunnions. § Two each.

The Burlington-Bristol Lift Span over the Delaware River is 533 ft 9 in. long. Light weight was obtained in this structure by the liberal use of silicon steel and a special light-weight floor. It was estimated that each pound added to the floor meant an additional cost of 12 cents in the trusses, towers, etc.

The 408-ft railway lift span, at Boonville, Mo., which carries the Missouri, Kansas, and Texas Railroad, was designed for E-70 loading.

The Hackensack River Crossing of the Delaware, Lackawanna, and Western Railroad, is a lift span 198 ft long. The maximum lift is 95 ft. To reduce the lifted load, an open deck was used, with silicon steel for the trusses and floor-beams.

The West Madison Street Bridge, in Chicago, Ill., is a double-leaf bascule. A total of 519 tons of nickel steel was used in the trusses and floor-beams.

The North Wabash Avenue Bridge, in Chicago, is a double-leaf trunnion bascule. In the trusses, 329 tons of nickel steel were used.

The South Harlem Avenue Bridge, in Chicago, is a double-leaf deck-riveted truss span, 224 ft, center to center of trunnions, and carries a 56-ft roadway and two 8-ft sidewalks. In the trusses, counterweight boxes, and two floor-beams, 812 tons of silicon steel were used.

The Outer Drive Bridge, in Chicago, is a double-leaf deck-riveted truss, 264 ft, center to center of trunnions, and carries two 38-ft roadways and two 14-ft sidewalks. The trusses and trunnion girders contain 1 965 tons of silicon steel.

Railroad Bridges

The Bessemer and Lake Erie Railroad High Bridge, over the Allegheny River, consists of two double-track continuous truss bridges. Silicon steel was used for the stringers, floor-beams, jacking girders, and for the main members of the trusses, except that the tension top chords over the piers were heat-treated eye-bars. The total weight of steel was 10 000 tons, of which 54% was silicon.

The three-span continuous truss bridge for the Chesapeake and Ohio Railroad, at Cincinnati, Ohio, is a double-track railroad bridge, 1 575 ft long. The center span is 675 ft and two side spans are 450 ft long. Silicon steel was used throughout at a unit stress of 24 kips per sq in. The resultant saving in steel weight was approximately 19% of the weight of a similar structure built of carbon steel using a unit stress of 18 kips per sq in. The decreased weight due to the use of silicon steel was particularly advantageous from an erection standpoint.

The railroad bridge across the Tanana River, for the Alaska Railroad, has a main span 700 ft long and 600 ft of plate-girder approaches. The live load was E-50. The item of freight was large and made the use of alloy steels attractive. Nickel-steel eye-bars were used on the 700-ft span, and silicon steel was used for all main members, the floor system, pins, and large detail parts. Silicon steel was also used for the flange angles of the 60-ft plate girders in the approaches.

The Atchison, Topeka and Santa Fé Railroad High Level Bridge, over the Illinois River, near Chillicothe, Ill., is a double-track structure, 1 696 ft long, consisting of a 400-ft and a 470-ft two-span continuous deck truss; a 440-ft through truss span; a 235-ft deck span; and two 68-ft plate-girder spans. The design live load was E-70. Silicon steel was used for all chords and main diagonal members, with carbon unit stresses increased 50% for tension and 40% for compression.

The Martinez-Benicia Bridge across Suisan Bay, in California, is a double-track railroad bridge, 5 600 ft long, built for the Southern Pacific Company. It consists of a 328-ft vertical lift span; seven through truss spans, 526 ft long; one deck span, 504 ft, and one deck span, 264 ft long; and a 780-ft approach viaduct. It was designed for E-90 loading, with unit stresses one-third higher than the standard. Silicon steel was used for the main members, except in the bottom chords where heat-treated eye-bars were used. The total weight of steel was 22 000 tons, of which 12 500 tons was silicon and 2 750 tons, heat-treated eye-bars.

The main river crossing of the Louisville and Nashville Railroad Bridge, over the Ohio River, at Henderson, Ky., consists of a double-track simple span, 670 ft long, and four double-track simple spans, 500 ft long. Silicon steel was used for the trusses, stringers, and floor-beams. The 670-ft simple span has a dead load of 14 kips per lin ft, and the total weight of steel in this span is 4 550 tons, of which 4 200 tons is silicon. The trusses are about 80% of the total weight of the span. The 500-ft simple spans weigh 2 500 tons each, of which 2 200 tons is silicon, and the trusses are about 67% of the total weight.

The Huey P. Long Bridge, over the Mississippi River, at New Orleans, La., is a combined double-track railroad and highway bridge (with one 18-ft roadway on each side). In the main spans and approaches 31 000 tons of silicon, steel was used. The 1 850-ft cantilever bridge and the five simple truss spans have a total weight of 20 000 tons, of which 11 000 tons are silicon steel and 1 400 tons, heat-treated eye-bars. The trusses, except minor members, are of silicon steel and heat-treated eye-bars. The railroad stringers and floor-beams are silicon, whereas the highway portion and the bracing are carbon. (For full-sized tests of heat-treated eye-bars for this bridge, see Table 15(*f*)).

Highway Bridges

The highway bridge over the Ohio River, at Louisville, Ky., consists of two 1 870-ft cantilever bridges, each composed of one anchor arm of 362 ft, one anchor arm of 500 ft, two 224-ft cantilever arms, and a 373-ft suspended span. It is 43 ft center to center of trusses and has two 6-ft sidewalks. The total weight of steel is 13 460 tons, of which 5 140 tons are silicon and 1 180 tons are heat-treated eye-bars.

A part of the Tri-Borough Bridge, at New York, N. Y., is a highway suspension bridge, having a 1 380-ft span and two 705-ft side spans. The 270-ft towers are of the fixed-base flexible type, having main columns of silicon steel with carbon-steel bracing. The stiffening truss chords are silicon steel, and the diagonals are carbon steel. The 96-ft floor-beams are of silicon. The total weight of silicon steel on the bridge is 15 000 tons. Each of the cables (20.63 in. in diameter) has 37 strands of 248 wires, 0.196 in. in diameter, and a total area of 277 sq in.

The Mount Hope (Providence, R. I.) Wire Cable Suspension Bridge has a 1 200-ft main span and two 504-ft side spans. The chords of the stiffening trusses are of silicon steel with working stresses of 34 kips per sq in. in tension

and 28 kips per sq in. in compression. The material in the two 11-in. cables, is the same as for the Ambassador Bridge, previously cited.

The main structure of the cantilever highway bridge across Carquinez Strait, in California, is 3 350 ft long and consists of two 1 100-ft main spans, a 150-ft central tower, and two 500-ft anchor arms. Silicon steel was used for the main material of the towers, and for the compression members and the built tension members of the trusses. Heat-treated eye-bars having an ultimate strength of 80 kips per sq. in. and a minimum yield point stress of 50 kips per sq in., were used for the principal tension members. The total weight of steel in the main bridge is 11 400 tons, of which 5 400 tons is silicon and 1 370 tons, heat-treated eye-bars.

The eye-bar cable suspension bridge, at Florianopolis, Brazil, has a main span of 1 113 ft 9 in., and carries a 28-ft roadway. In this bridge a part of the cable, which is composed of heat-treated eye-bars, acts as the center part of the top chord of the stiffening truss. The heat-treated eye-bars were specified to have a minimum ultimate strength of 105 kips per sq in., a minimum yield point stress of 75 kips per sq. in., and an elongation of 5% in 18 ft.

The eye-bar suspension bridges over the Ohio River, at Point Pleasant and St. Marys, W. Va., have a 700-ft main span and two 380-ft side spans and carry a 22-ft roadway and one 5-ft sidewalk. These bridges are similar to the Florianopolis Bridge, except that in addition to the eye-bars replacing the twelve center panels (of twenty-eight panels) in the main span, they replace the first seven panels from the cable bent (of fifteen panels) in the side spans. The physical characteristics of the special heat-treated eye-bars in this bridge are given in Table 15(*g*).

The self-anchored, chain, suspension type of bridge was used on the 6th, 7th, and 9th Street Bridges over the Allegheny River, at Pittsburgh, Pa. The center span is 442 ft long, with two side spans of 221 ft. Each chain was made of eight and nine eye-bars alternating with a maximum section of 230 sq in. These eye-bars were heat-treated to a minimum yield point stress of 50 kips per sq in. and an ultimate strength of 80 kips per sq in.

The Waldo-Hancock Bridge, in Maine, is cited as an example of the twisted-wire strand cable, suspension bridge. It has a main span of 800 ft, and two side spans of 350 ft. Each $9\frac{5}{8}$-in. cable consists of 37 strands of $1\frac{3}{8}$-in. wire rope. The ultimate strength per strand is 216 kips and each strand was pre-stressed to one-half this amount. The modulus of elasticity, after pre-stressing, was 26 600 000 lb per sq. in.

Buildings

Special steels have not been used extensively in tier buildings. In one of the buildings of Radio City, in New York, N. Y., silicon steel was used in a part of four columns in order to reduce the size.

The Tower Building for the Cleveland Terminal Company, in Cleveland, Ohio, is a 52-story building, being about 300 by 247 ft for 16 stories, and then a set-back tower, 489 ft high, has a base that is 98 by 98 ft. The total weight of steel was 16 850 tons, of which 4 610 tons was silicon.

The hangar built for the Goodyear Zeppelin Corporation at Akron, Ohio, is 325 ft wide by 1 184 ft long ("out-to-out" of doors) by 180 ft clear height, having eleven lattice arches spaced on 80-ft centers and two diagonal half arches at each end. Silicon steel was used for the main arch ribs only. Of the total weight of 7 210 tons, 628 tons is silicon.

In the Richmond Power Station, at Philadelphia, Pa., there are 10 000 tons of steel in the switch house, turbine hall, and boiler house, of which 8 000 tons is silicon. Silicon steel was used for all the principal members of the main building framework, except the framing of the turbine hall roof. Tests from sixty-five heats are listed in Table 15(*h*).

The United States Post Office Building, in Philadelphia, Pa., covers an area of 380 by 460 ft, and has five floors and a roof. The total weight of steel is 17 700 tons, of which 1 740 tons is silicon. Silicon steel was introduced in this building when two hundred and fifty-two 36-in. silicon beams without cover-plates were substituted for carbon beams with cover-plates.

The addition to the Post Office Building, at Washington, D. C., is an irregular-shaped structure (width, 230 to 120 ft; and length, 418 to 368 ft; height, six floors and roof). The total weight is 7 180 tons, of which 2 975 tons is silicon. Silicon steel was used for the heavy plate girders, the double girders, part of the column slabs, part of the floor-beams (usually 24 in. and deeper) and about 62% of the trusses and columns.

Special Structures in Which Transportation or Other Costs are Factors in Making the Use of Special Steels Economical

One of the best examples is the steel derrick used in the erection of steel bridges. The additional cost of the special steel is more than compensated for in three ways: First, the saving in weight reduces transportation costs as the derrick is shipped to and from the tool house to the bridge sites; second, the larger capacity; and third, the saving in weight in the steel of the bridges due to smaller erection stresses.

Silicon steel derricks were designed for use on the San Francisco-Oakland Bay Bridge. With a 103-ft mast and an 85-ft boom, one of these derricks weighed about 32 tons and made lifts of 59 tons. A similar carbon steel guy derrick weighs about 34 tons and has a lifting capacity of 46 tons. For slightly less weight a silicon steel derrick will lift about 40% more load.

The traveling crane is another type of special structure in which saving in weight, due to the use of high tensile steels, more than pays for the additional cost, because power consumption is reduced. The writer knows of three mill types of cranes, 120-ft span, which have a 40-ton main hoist and a 15-ton auxiliary hoist. The main girders are riveted plate girders. A reduction in weight of 40 000 lb is due to the use of manganese steel. It was necessary to deepen the girders 1 ft (compensating for the higher unit stresses on a material having the same modulus of elasticity as the carbon steel) in order to keep the deflection the same as for the carbon-steel cranes. It is generally not economical to use special steel unless the crane span is more than 100 ft.

The single-track railroad viaduct of the Pittsburgh and West Virginia Railway Company near Banning, Pa., was being built on a new line, so that additional erection equipment could not be readily sent to the field. In this viaduct there were two 120-ft carbon-steel plate girders, weighing 65 tons each, the weight and reach of which were beyond the capacity of the erection derrick. Permission was obtained to design the girders for the use of silicon steel. The silicon girders weighed 53 tons each and could be handled by the derrick.

Alloy steels are seldom used in mill building, but in the Open-Hearth Building for the National Tube Company, at McKeesport, Pa., some of the crane girders were 115 ft long. The type of erection rig used to construct mill buildings does not have the capacity to handle long heavy girders. The girders, in this case, were designed for silicon steel, thus reducing the weight to 93 tons which could be handled by two locomotive cranes.

Bridges in the United States have been growing longer and heavier, and with this growth have come increased stresses and larger field rivets. Because of the stiffer material and more thicknesses it is becoming increasingly difficult to draw connecting members tight for proper rivet driving. The erectors find it necessary to use pneumatic tools to tighten fitting-up bolts, which strip the thread or break the ordinary carbon-steel bolt. Manganese steel fitting-up bolts which are now used extensively, obviate this difficulty and can be utilized over and over again. Even the hand wrenches are being made of manganese steel. The specifications for this material are given in Table 15(*i*).

All bars forming grills for the tunnel intake for the New Kenawha Power Company, at Hawks Nest, W. Va., were special Cromansil steel welded to the main frame of carbon steel. It is of interest to note the latitude of the specifications (see Table 15(*j*).

A great variety of special forging steels and a considerable tonnage of stainless steel are used in the movable dams constructed for the Federal Government. Abrasion-resisting steel is also used in the structural steel work of chutes for the U. S. Coke and Coal Company.

Copper-bearing steel is coming into more general use on account of its corrosion-resisting properties. In some cases, particularly for the Louisville and Nashville Railroad Company, it is specified for steel in the floor of bridges and other parts likely to be affected by brine from refrigerator cars. In many cases, such as the San Francisco-Oakland Bay Bridge, it is specified for all parts of the structure. A minimum of 0.20% copper is usually specified.

Wire rope is used not only for main cables and hangers on suspension bridges, but also for hangers on several riveted bridges, such as the Bayonne, N. J., Bridge and the West End-North Side Bridge, in Pittsburgh, Pa. It is also used for counterweight and operating ropes on lift bridges.

It has recently become necessary, in certain work, to design and detail structural steel so that it can be transported by airplane. The usual requirements are that the weight of a piece shall not exceed 2 000 lb and that the

piece can be loaded through a 3 by 7-ft door. The advantage of the saving in weight which is offered by the use of special steels, is apparent in such cases.

Shop Practice for Special Structural Steels

The fabrication of special steels has brought new problems to the shops because equipment was designed and powered for handling carbon steel. The equipment or capacity to fabricate the high tensile steels varies with each individual plant. The following discussion of shop practice is based on the typical or general equipment, and it must be realized that each plant may have some one tool with more power or capacity.

Cambered plates in heavy gages cannot be straightened with straightening rolls or in a press. Camber is removed from the large thick plates by planing the edges. The strength of high tensile steels is such that the usual fabricator is not equipped to straighten it as he would straighten rolled carbon steel. Efforts to cold-straighten heavy plates ($\frac{3}{4}$ in. and more) have produced badly cracked edges. One large fabricator prefers to divide plate thicknesses greater than $\frac{7}{8}$ in. into two thinner plates stitch-riveted together.

Some plates, $1\frac{1}{4}$ in. and more in thickness, cracked at the sheared ends when an attempt was made to flatten a slight shear bow by passing the plates through cold rolls. This bow was caused by the pressure of the movable shear blade on the outer ends of the plates where they were not supported by the fixed blade or die. The difficulty was overcome by planing $\frac{1}{4}$ in. from sheared ends and straightening the plates in the press. Subsequently, the bowing was avoided by turning the plates around so that in cutting both ends, the main body of the plate rested on the die of the shear.

As piping is deeper in silicon ingots than in carbon steel, angles should be ordered long enough for a preliminary shearing (or milling) at each end, in search of pipes.

It has been reported that the silicon plates for the towers of the Golden Gate Bridge were sub-punched with fewer punches broken than the plates for the towers of the Philadelphia-Camden Bridge. The records show that the average of the latter was as given in Table 15(k), whereas, the former, at the time the report was made, was as shown in Table 15(l).

A test has been devised which records on an indicator card the force of the punch, simultaneously with its penetration into the plate. Other tests have been made on power input required to drill and plane. Plates were used which had passed silicon steel acceptance tests and were of about the same strength. One analysis was: Carbon, 0.26%; manganese, 0.84%; and silicon, 0.27%; and the other was: Carbon, 0.35%; manganese, 0.84%; and silicon, 0.25 per cent. The former punched with 10% less maximum pressure than the latter. In drilling and planing, the difference was nearly the same, and in the same direction.

In 1929, a group of twenty-four types of high tensile steels in various structural shapes were studied to determine which of the different grades could be fabricated successfully with the present bridge shop tools. As described in Table 15(m), all these steels were in the manganese-silicon

group and represented various combinations of these elements with carbon. In addition, there were a few sections containing about 2% nickel. The sections included $\frac{1}{2}$-in. and 1-in. plates, 8 by 8 by $\frac{3}{4}$-angles, 12-in., 40.8 lb I-beams, and 14-in., 425-lb C-beams. These sections were subjected to various bridge shop operations which included punching, drilling, reaming, milling, planing, and coping, with the results noted in Table 17. Test coupons were cut from each item and tensile, bending, and hardness tests were made (see Table 15(*m*)).

TABLE 17.—FABRICATION OF HIGH STRENGTH STRUCTURAL STEELS WITH PRESENT SHOP TOOLS

Item No. (see Table 15 (*m*):	Punching	Drilling	Reaming	Rotary planing	Milling	Coping	Edge planing
28	*a, d*	*a*	....	....	....	....	....
29	*a, d*	*a*	....	....	....	....	....
30	*a, d*	*a*	....	....	....	....	....
31	*b*	*b*	*b*	*b*	....	....	....
32	*b*	*b*	*b*	*b*	....	....	....
33	*b*	*b*	*b*	*b*	....	....	....
34	*a d*	*c*	....	*a*	....	....	*a*
35	*a d*	*c*	....	*a*	....	....	*a*
36	*e*	*e*	*e*	*a*	....	....	....
37	*e*	*e*	*e*	*a*	....	....	....
38	*e*	*e*	*e*	*a*	....	....	....
39	*a*	*a*	....	....	*a*	....	....
40	*a*	*a*	....	....	*a*	....	....
41	*f*	*f*	....	....	*f*	....	....
42	*f*	*f*	....	....	*f*	....	....
43	*f*	*f*	....	....	*f*	....	....
44	*f*	*f*	....	....	*f*	....	....
45	*g*	....	....	....	....	*h*	....
46	*g*	....	....	....	....	*g*	....
47	*g, d*	....	....	....	....	*h*	....
48	*g, d*	....	....	....	....	*h*	....
49	*g, d*	....	....	....	....	*h*	....
50	*d*	....	....	....	....	*g*	....
51	....	*i*	....	*a*	....	....	....

[a] Considered impracticable; too difficult. [b] Satisfactory at speeds and feeds slightly less than those required for silicon steel. [c] Not quite as unsatisfactory as Item No. 28. [d] The shock to the punching machine was terrible. [e] Satisfactory at speeds less than Item No. 31. [f] Similar to Item No. 36. [g] Fair results. [h] Flanges split. [i] Practical at very low speeds.

In this general discussion, it should be recognized that two steels of the same ultimate strengths do not necessarily possess the same ease of fabrication, and that with the newer steels, current knowledge of this phase is quite incomplete.

Shearing.—Shears have been rated and powered for carbon steel, and it is not practical to reduce the speed. Consequently, with the higher tensile steels the capacity is correspondingly reduced. For silicon and manganese steels the shears are rated at approximately 75% capacity; for example, a large shear is rated to shear a carbon-steel plate, 1 in. thick by 100 in. wide. This same shear will safely shear a silicon plate, $\frac{3}{4}$ in. thick by 100 in. wide. The shear blades are not changed for the alloy steels, since the blades used for carbon steel are the hardest obtainable.

Punching.—The old "rule of thumb" for punching carbon steel was that the material should not be thicker than the size of the hole to be punched. With silicon steel and manganese steel, whether sub-punching or full-sized punching, and regardless of whether the holes are $\frac{13}{16}$ in. in diameter, or larger.

the shop prefers a maximum thickness of $\frac{3}{4}$ in., whereas for nickel steel the preferred maximum is $\frac{1}{2}$ in. Heavier material than this puts too great a strain on the machines. When punching alloy steels, the speed of the punches is usually reduced about 25 per cent.

It is a recognized fact that, with variations in composition, alloy steels may vary in hardness, and that some melts punch more easily than others. Where the specifications permit full-size punching, the shop will try to punch silicon and manganese to a maximum thickness of $\frac{7}{8}$ in. and nickel steel to a maximum of $\frac{9}{16}$ in. If the metal is too hard, as shown by the breaking of the punches or the strain on the machine, the holes will be drilled.

Because of the excessive breaking of the punches due to the harder alloy steels they have been re-designed. Larger fillets are used where the section of the punch changes radically. The stress concentration at this point was accountable for 50% of the punch breakage.

It is desirable to maintain scrupulously, or if possible exceed, the edge distance (and rivet-center distances) ordinarily specified. With 1-in. rivets, no angles with legs less than 4 in. should be used in the design.

One of the items that causes additional punching costs is the special handling due to punching holes of two sizes in the same plate because sizes of shop and field rivets are not properly proportioned, or the amount of reaming varies for shop and field rivets.

TABLE 18.—Drilling Speeds

Description	Peripheral Speed of Drill, in Feet per Minute				Speed of Reamers, in Revolutions per Minute									
					Reaming Sub-Punched Holes to Full Size					Spear Reaming Full-Sized Holes				
	Medium Open-Hearth Steel		Silicon Steel		Medium Open-Hearth Steel		Silicon Steel		Nickel steel	Medium Open-Hearth Steel		Silicon Steel		Nickel steel
	From:	To:	From:	To:	From:	To:	From:	To:		From:	To:	From:	To:	
Plant No. 1:														
Gantries......	50	70	45	60	170	260	150	240	.....	240	280	235	275	......
Multiple spindle.....	70	85	65	80	....	...	....	...	.....	.....	.....	....	.....	......
Radials.......	50	60	45	55	160	270	150	250	.....	.....	.....	.....	.....	......
Plant No. 2:														
Medium open-hearth steel.	....	90	....	..	....	...	....	...	.....	.....	.....	.....	.....	......
Silicon steel...	70	75	....	..	....	...	....	...	.....	.....	.....	.....	.....	......
Nickel steel...	50	55	....	..	....	...	....	...	.....	.....	.....	.....	.....	......
Hi-cycle......	....	..	....	..	270		270		220	270		270		220
Hi-cycle......	....	..	....	..	250		250		160	250		250		160

Drilling from the Solid.—With the harder alloy steels drilling from the solid is a matter of speed and feed. Thickness is not a factor unless the plates are stacked. The shop practice of sub-drilling, assembling the member, and reaming holes to size, or of assembling the member and drilling holes full sized, varies with the fabricating plants and also varies in an individual shop; it is determined by the types of work being fabricated.

The reduction in speed of the machines when drilling special steel can be seen from the report of plants as shown in Table 18.

The drilling of alloy steels requires more grinding of drills, which also entails the added cost of changing the drills more often. One plant reports that, in comparison with ordinary carbon steel, drills used in silicon and manganese steels must be ground somewhat more often, and those used in nickel steel, twice as often. One plant reports on nickel-steel bridge chords (in which the number of holes to be drilled per member averaged 539, with a thickness of $2\frac{5}{8}$ in.) that the best record on one chord required the drills to be changed fifty-five times, whereas on one chord the drills had to be changed ninety-seven times. For comparison purposes, they estimated that on a similar chord of carbon steel they would have required eight to ten drill changes.

The feed of drilling varies with the different alloy steels and even on one piece of angle or plate, and can only be stated as being less than for carbon steel.

Reaming.—Reaming of holes is more universally specified for alloy steels than for carbon steels. It will be noted from Table 18 (a report on the reduced speeds necessary with special steels) that the reduction is not as marked as noted for drilling.

Milling.—Planing speeds for silicon, manganese, and nickel steels are generally reduced about 25%, and the feeds are changed to suit the particular hardness of each metal. It is difficult to get a smooth finish on these alloy steels on account of the tool wear.

Chipping.—The increased difficulty of chipping silicon and manganese steels may be covered by stating that the speed is reduced about 25 per cent. Nickel steel is extremely difficult to chip under any conditions and depends on previous treatment, whether sheared or cold sawed; if cold sawed, it is almost impossible to chip, and may require pre-heating.

Flame Cutting.—Flame cutting of carbon steel of structural grades leaves a perfectly ductile edge for milling or chipping. If silicon steel is flame cut the edge is air-hardened, and it is difficult to edge-plane or mill. However, a milling cut of $\frac{5}{32}$ in. will remove all trace of damage. The portion affected by the flame varies in hardness and although the piece usually can be edge-planed without serious difficulty after a cutting torch is used near the surface to be planed, it is sometimes necessary to pre-heat it. Manganese and nickel steels offer more difficulty. If either is flame cut, it cannot be planed (without special treatment that is not practical).

Experiments with a special portable heat-treating and flame-cutting machine indicate the possibilities of flame cutting some of the alloy steels (the standard nickel, manganese, and silicon types), leaving an edge of sufficient ductility for further shop operations. The results of the bend tests and drift tests indicate that the process will meet any required specifications.

Bending.—In discussing alloy steels it is usually stated that the radius required to bend a plate cold depends upon the thickness of the plate, as expressed by:

$$r = C\,d \qquad (4)$$

in which r = the radius of bend; C = a coefficient; and d = depth, or thickness of the plate.

The writer has noted, in various papers, that, for plates with $d = \frac{3}{4}$ in., or less, $C = 2$ for Cor-Ten; 2.5 for Man-Ten; and 3.0 for Cromansil.

The radius required for bending a plate cold should vary with the kind of steel, the thickness of the plate, and the angle of bend. Table 19, which

TABLE 19.—MINIMUM RADII, r, IN INCHES, FOR BENDING PLATES AND ANGLES

Type of stee	Plates				Angles; Bent Cold		
	Thick-ness of plate	Bent Cold		Bent hot; bends greater than 90°	3 by 3-in. to 4 by 4-in.	6 by 6-in.	8 by 8-in.
		45° bend	90° bend				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Silicon	0.375	0.375	0.625	0.875	36.0	48.0	120.0
	0.625	0.625	1.5	1.75			
	0.875	0.875	2.625	2.875			
	1.125	1.125	3.625	4.0			
Nickel	0.375	0.5	0.75	1.0	42.0	54.0	120.0
	0.625	0.75	1.625	1.875			
	0.875	1.00	2.75	3.0			
	1.125	1.25	3.75	4.25			

illustrates these variables gives the minimum radii required for bending silicon and nickel-steel plates. These radii are to be used when the bend is transverse to the plate. When the bend is longitudinal (that is, when the bend line is parallel to the direction of rolling the plate) these radii should be increased slightly.

Assembling.—The increased stiffness of the special steels makes it more difficult for the fitting-up bolts to draw the steel together into perfect contact, but this close contact is necessary for the driving of satisfactory rivets. The smallest fitting-up bolts which the shop should use are $\frac{3}{4}$ in. in diameter. This means that the minimum diameter of the holes should be $\frac{13}{16}$ in. Where sub-punched and reamed work is specified, the minimum shop rivet should be 1 in. in diameter. The increasing use of alloy steels and the difficulties of fitting up properly have necessitated the use of stronger fitting-up bolts, and the fabricating plants are beginning to make them of manganese steel. One plant now uses 100% manganese bolts for all bridge work.

Rivets.—The manufacture of carbon-steel rivets and their use in fabricated steel structures has become standardized. The need for a rivet of higher strength to match the special steels was recognized. Nickel-steel rivets with a nickel content of 3 to $3\frac{1}{2}$% have been used on a few structures, but have been so invariably unsatisfactory from every standpoint that they should not be specified for any structural steel work. However, tests have been made on rivets that meet Steel Specification No. 2115 of the Society of Automotive Engineers (carbon, 0.10 to 0.20%; manganese, 0.30 to 0.60%; silicon, 0.15 to 0.30%; and, nickel, 1.25 to 1.75%) which indicate that this grade is suitable for hot riveting and will develop a shearing strength comparable to that of other high-strength rivets in use to-day. Whereas silicon steel is admittedly not a forging steel and not satisfactory for rivet material, manganese steel

has been found to be satisfactory both for manufacturing and driving. Shipbuilders have found that manganese steel makes a good high-strength riveting material. They have used it in large quantities and have had considerable research work done to establish the best chemical composition.

A large number of manganese rivets was used on the Bayonne Bridge (see Table 15(e), Item No. 2). The maximum rivet was $1\frac{1}{4}$ in. in diameter by $10\frac{3}{4}$ in. long. The rivets were tapered if the length was five or more times the nominal diameter. Later experiments indicated that this rivet material was a little too hard, and a more satisfactory material has been developed (see Table 15(d)). For the San Francisco-Oakland Bay Bridge the largest manganese rivets were $1\frac{1}{4}$ in. in diameter by $10\frac{1}{2}$ in. long, $1\frac{1}{8}$ in. in diameter by $9\frac{1}{2}$ in. long, and 1 in. in diameter by $6\frac{3}{4}$ in. long. These rivets were tapered the same as for the Bayonne Bridge.

The driving technique for manganese rivets differs considerably from that for carbon rivets. The more important differences are: (1) Manganese rivets must be more carefully heated; (2) they are more easily burned than ordinary rivets; (3) the range of temperature is small between a proper driving heat and a burning heat; (4) more time is required in driving than for ordinary rivets; (5) the cost of driving is estimated at 10% greater; (6) it is necessary to burn off the heads before backing condemned rivets out; (7) it costs about twice as much per rivet to remove them; and (8) with the proper attention to heating and driving there will be no more cut-outs than for ordinary rivets. There is a need for a standard specification for high-strength structural rivets.

The action of riveted joints is a controversial subject and some criticism of manganese rivets has been offered because the tests show that they have an initial slip at lower values than carbon rivets. However, the point should be brought out that the factor of safety is not based on the initial slip value, but on the relation of the working stresses of the material and details, such as rivets, to the yield points, and upon the actions of these materials under laboratory tests and actual use, when stressed between these limits. Tests made for the Bayonne Bridge showed that carbon rivets averaged from 0.004 in. to 0.008 in. more slip than the manganese rivets. Other tests have shown a contrary result.

In order to maintain the factor of safety for the structure, the rivet factor must be maintained on an equal basis with the main material between the working stress and up to the yield point. Taking an average of all available tests, it seems fair to state that the slip for manganese and carbon rivets will be of approximately equal magnitude through the range of stress in which the designer is interested. At ultimate values the manganese rivets have a larger factor of safety than carbon rivets.

In 1906, tests of nickel and chromium steels were reported at the University of Illinois[79] with the following conclusions:

(a) The ultimate shearing strength of rivet joints depends on the shearing strength of rivet material, and this is influenced by the relative hardness of rivets and plates;

---

[79] "Tests of Nickel-Steel Riveted Joints", *Bulletin*, Vol. 8, No. 26, Univ. of Illinois, Urbana, Ill.

(*b*) The ratio of the yield point stress in riveted joints to ultimate shearing strength of riveted joints is about the same as the ratio of the yield point stresses of plates in tension to the ultimate strength of the plate.

(*c*) In riveted joints designed on the basis of ultimate strength, the strength of the rivet material and of the plate material is of prime importance, and use of special steels of great strength may be of advantage.

(*d*) In riveted joints designed on the basis of the frictional hold of rivets there is little advantage in using rivets of special steel since joints with such rivets show about the same resistance to first noticeable slip as joints with ordinary carbon rivets; and,

(*e*) There was a noticeable slip of the joint, generally, at loads within the ordinary working shearing stress of rivets.

Tests by Commander E. L. Gayhart[80], U. S. N., were made on high-strength steels and rivets. He stated that joints fabricated with high tensile steel showed resistance to slip greatly inferior to that shown by medium steel—values so low that it appears doubtful whether any great part of the load could be considered as carried by frictional forces. In terms of single shear stress, according to Commander Gayhart, the stress in the rivet at which the first slip occurs is so much less than customary design shear stresses, and is so greatly affected by the number of rivets in the joint, that it does not appear desirable to design riveted joint construction on the basis of frictional resistance but rather by the commonly used method of proportioning on shearing and bearing.

As the designer depends on the bearing of the rivet on the plate to carry the stress, there is no advantage in using high-strength rivet steel in carbon plates; for example, on the Bayonne Bridge a bearing value was used for the manganese rivets, of 45 kips per sq in., on manganese plates; 40 kips per sq in. on silicon plates; and 30 kips per sq in. on carbon plates. As 30 kips per sq in. is also used on bearing carbon rivets on carbon plates, it is evident that the value of a manganese rivet and a carbon rivet in bearing on a carbon plate is the same.

Welding.—The scope of this paper does not permit the introduction of very much material on the subject of welding for to the writer's knowledge no structural steel bridges or buildings have been constructed in which the special steel was welded to carry stresses. However, considerable welding on special steels is being done in the plants. Pre-heating and annealing of large structural members is not practical; practically all the welding is done by the metallic arc.

Considerable confusion exists at present because there is no standard or definition stating what chemical or physical qualifications a steel must have in order to be classed as of weldable quality. The various steel companies list their special steels as those that can be welded, whereas the specifications of the American Society for Testing Materials do not list them as "steel suitable for fusion welding." On one hand the United States Navy

[80] "An Investigation of the Behavior and of the Ultimate Strength of Riveted Joints Under Load", by Commander E. L. Gayhart, U. S. N., *Proceedings*, Soc. of Naval Archts. and Marine Engrs., Vol. 34 (1926), p. 55.

Department is quoted as being definitely against the use of the silicon-manganese types of steels for welded stress-carrying members; and yet U. S. Navy Specification No. 48S 5e specifies (under high tensile steel of weldable quality) a steel with the characteristics listed in Table 15(*n*).

Probably the most interesting point to be emphasized in the welding of special steels (for the purpose for which welding is used in structural shops) is that generally the ordinary soft steel, coated, welding rods are used rather than the special rods developed by various manufacturers. The technique required for the ordinary coated rods is much simpler, and the strength developed is more than ample for the fillet welds.

Tension tests were made on full butt welds in manganese-steel plates using several of the special welding rods and all the welds developed more strength than the parent metal. However, many of the special steels are subject to "air-hardening" or grain growth, so that if it is necessary to bend the steel, or if it is subject to bending stresses, the air-hardened surface must not be located on the tension side, unless the steel can be annealed, which is usually not practical for structural steel work.

Welding of special steels in the fabricating shops has generally been confined to: (1) Tack welding for assembling; (2) fillet welding fillers for permanent work; and (3) some welding of silicon slabs for shoes, with special supervision. Covering this type of welding, it is stated[77] that in the fabrication of the towers for the Golden Gate Bridge, it was desired to tack-weld the shaft angles and the web-plates so that the full thickness could be drilled from the solid steel simultaneously, with no subsequent separation before riveting. The question was raised as to whether this tack-welding would embrittle the silicon steel harmfully, which was of the composition listed in Table 15(*l*). Specimens, therefore, were tack-welded, broken apart, and bent; other specimens were drilled and bent until cracks appeared around the drilled holes. The broken weld edges would bend farther than the drilled holes before cracks appeared. Other plates were tack-welded on the edges, drilled, riveted, and bent. The rivets sheared before any cracks appeared in the welds. It was proved, therefore, that the tack-welding was not detrimental to the structure in comparison with the other acceptable processes of fabrications. This was not intended to, and did not, touch on the question of welding for transmission of stress.

Stress-strain data for the silicon steel used in the towers of the Golden Gate Bridge are listed in Table 20, the average chemical composition (previously discussed) being given in Table 15(*l*). Specifications for standard mill tests required that the yield point, or the drop of the beam, should be equal to, or greater than, 45 kips per sq in. The proportional limit was defined as the point on the stress-strain curve at which the rate of deformation is 50% greater than it is at the origin.

Sketches of the test coupons, and their standard position in plates and angles, are shown in Fig. 33. These specimens were made about 0.505 sq in. in diameter, except for material 0.5 in. thick, or less; in the latter case the

TABLE 20.—CHECK TESTS ON SILICON STEEL IN TOWERS, GOLDEN GATE BRIDGE

Test No.	Metal from top or bottom of ingot* (1)	Dimensions, in inches (2)	UNIT STRESSES, IN KIPS PER SQUARE INCH: Yield point (3)	Proportional limit (4)	Ultimate strength (5)	Modulus of elasticity, in millions of pounds per square inch (6)
(a) SHEARED PLATES (SEE FIG. 33 (a))						
1	Top	15/16 in. thick	53.5	48.5	94.4	28.9
1	Bottom	15/16 in. thick	48.5	48.5	82.0	29.5
2	Top	7/8 in. thick	55.0	55.0	90.3	29.2
2	Bottom	15/16 in. thick	50.0	50.0	82.1	28.2
3	Top	15/16 in. thick	51.3	51.3	89.9	27.4
3	Bottom	7/8 in. thick	47.5	44.3	87.6	26.5
4	Top	7/8 in. thick	52.2	50.9	93.0	28.7
4	Bottom	7/8 in. thick	52.0	50.0	91.5	30.0
5	Top	7/8 in. thick	45.0	41.2	87.0	28.4
5	Bottom	7/8 in. thick	45.0	41.2	85.6	30.4
6	Top	7/8 in. thick	50.0	48.8	87.4	29.9
6	Bottom	7/8 in. thick	47.2	44.8	87.8	29.7
7	Top	7/8 in. thick	57.5	53.7	97.1	30.2
7	Bottom	7/8 in. thick	52.2	51.6	85.0	28.8
8	Top	5/8 in. thick	51.3	51.3	88.2	29.5
8	Bottom	15/16 in. thick	42.5	38.0	81.6	30.0
9	Top	13/16 in. thick	43.8	35.6	87.1	29.6
9	Bottom	13/16 in. thick	42.6	37.5	83.9	28.0
10	Top	7/8 in. thick	51.3	42.5	95.1	28.4
10	Bottom	1/2 in. thick	52.0	52.0	84.9	29.2
11	Top	3/4 in. thick	47.2	44.8	83.4	28.3
11	Bottom	3/4 in. thick	46.0	46.0	83.7	30.1
(b) UNIVERSAL MILL PLATES (SEE FIG. 33 (a))						
12	Top	15/16 in. thick	46.0	42.6	83.4	29.3
12	Bottom	15/16 in. thick	46.0	43.3	84.8	29.4
13	Top	15/16 in. thick	49.7	46.9	88.6	29.3
13	Bottom	15/16 in. thick	48.5	47.2	80.3	30.1
14	Top	15/16 in. thick	43.8	35.0	86.4	28.1
14	Bottom	1/2 in. thick	47.9	47.3	82.7	28.3
15	Top	7/8 in. thick	47.1	43.6	84.3	30.2
15	Bottom	7/8 in. thick	45.8	45.8	86.0	30.2
16	Top	7/8 in. thick	48.5	43.5	91.3	30.2
16	Bottom	7/8 in. thick	47.1	42.6	85.3	29.8
17	Top	5/8 in. thick	50.0	48.7	88.2	30.3
17	Bottom	7/8 in. thick	47.2	43.4	88.4	30.9
18	Top	13/16 in. thick	51.0	41.2	93.4	31.2
18	Bottom	3/4 in. thick	(†)	39.2	88.8	30.1
19	Top	3/4 in. thick	50.0	45.0	93.0	28.8
19	Bottom	5/8 in. thick	46.4	42.8	81.6	27.5
20	Top	5/8 in. thick	50.0	48.8	92.0	27.6
20	Bottom	5/8 in. thick	47.5	45.1	85.3	26.9
21	Top	5/8 in. thick	51.5	42.7	93.9	29.6
21	Bottom	1/2 in. thick	50.8	41.2	87.6	31.8
22	Top	1/2 in. thick	50.0	43.3	88.5	25.9
22	Bottom	1/2 in. thick	50.0	41.0	86.8	26.7
(c) ANGLES (SEE FIG. 33 (b))						
23	Top	8 by 8 by 5/8	(†)	34.0	86.6	32.0
23	Bottom	8 by 8 by 7/8	42.5	36.6	82.8	28.1
24	Top	8 by 8 by 5/8	46.3	37.5	87.8	33.4
24	Bottom	8 by 8 by 7/8	48.5	48.1	87.4	29.1
25	Top	8 by 8 by 5/8	(†)	46.4	99.1	30.6
25	Bottom	8 by 8 by 5/8	(†)	45.0	96.5	31.4
26	Top	8 by 8 by 5/8	51.0	42.3	94.9	32.7
26	Bottom	8 by 8 by 5/8	46.0	40.2	84.1	28.7
27	Top	8 by 8 by 5/8	50.0	38.5	89.4	31.3
27	Bottom	8 by 8 by 5/8	48.9	35.1	89.1	30.8
28	Top	8 by 8 by 5/8	43.8	33.4	82.1	30.5
28	Bottom	8 by 8 by 5/8	45.0	30.9	82.1	31.3
29	Top	8 by 8 by 5/8	(†)	35.0	89.6	31.3
29	Bottom	8 by 8 by 7/8	47.5	47.1	85.8	29.1
30	Top	6 by 6 by 11/16	(†)	37.3	89.9	29.0
30	Bottom	6 by 6 by 11/16	(†)	36.3	84.8	32.4
31	Top	6 by 6 by 11/16	(†)	33.0	89.4	29.4
31	Bottom	6 by 6 by 11/16	(†)	36.1	88.7	29.6

TABLE 20.—(*Continued*)

Test No.	Metal from top or bottom of ingot* (1)	Dimensions, in inches (2)	Unit Stresses, in Kips per Square Inch: Yield point (3)	Proportional limit (4)	Ultimate strength (5)	Modulus of elasticity, in millions of pounds per square inch (6)
(*d*) Flats (See Fig. 33 (*a*))						
32	Top	7 by 1/2	54.3	35.1	100.1	28.3
32	Bottom	7 by 1/2	53.8	51.5	93.8	28.9
33	....	5 1/2 by 7/8	54.7	53.5	91.8	30.5
34	....	7 by 3/4........	(†)	40.0	104.5	31.1
35	....	6 by 3/4........	57.5	56.6	93.0	29.3
36	....	6 by 3/4........	57.5	55.0	94.9	27.4
37	....	6 by 5/8	52.6	50.7	96.8	29.3
38	....	3 1/2 by 5/8	57.6	56.4	91.3	29.3
39	....	3 1/2 by 5/8	52.5	51.3	85.7	31.3
40	....	3 1/2 by 5/8	56.4	53.9	90.9	27.5
41	....	8 by 1/2	54.8	54.8	93.3	29.4

* Not necessarily from the same ingot in any test. † Indefinite yield point.

specimen was turned to the largest diameter possible. Stress-strain curves for a series of comparative tests of silicon and nickel steel used in the Delaware River Bridge are given in Fig. 34, the corresponding chemical and

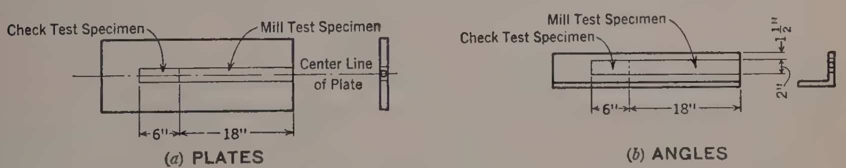


Fig. 33.

physical characteristics for this series being listed in Table 15(*o*). Fig. 35 illustrates the type of specimen tested in Table 15(*o*) and in Fig. 34. The letters, *C*, *B*, and *A*, indicate the location of coupons corresponding to Items Nos. 63, 64, and 65, respectively, in Fig. 34.

Heat-Treated Eye-Bars

Heat-treated eye-bars are made in two grades: (1) Grade HT—high tensile strength (Table 15(*f*)); and (2) Grade HT Special—special high tensile strength (Table 15(*g*)). When acceptance of eye-bars depends on the results of full-sized tests, the standard requirements are as shown in Table 21. When chemical and physical properties are specified, the following standards

TABLE 21.—Specifications Based on Full-Sized Tests

Description	Grade HT*	Grade HT Special†
Tensile strength, minimum, in kips per square inch................	80	105
Yield point, minimum, in kips per square inch.....................	50	75
Percentage elongation in 18 ft., minimum..........................	8	5

* See Table 15 (*f*). † See Table 15 (*g*).

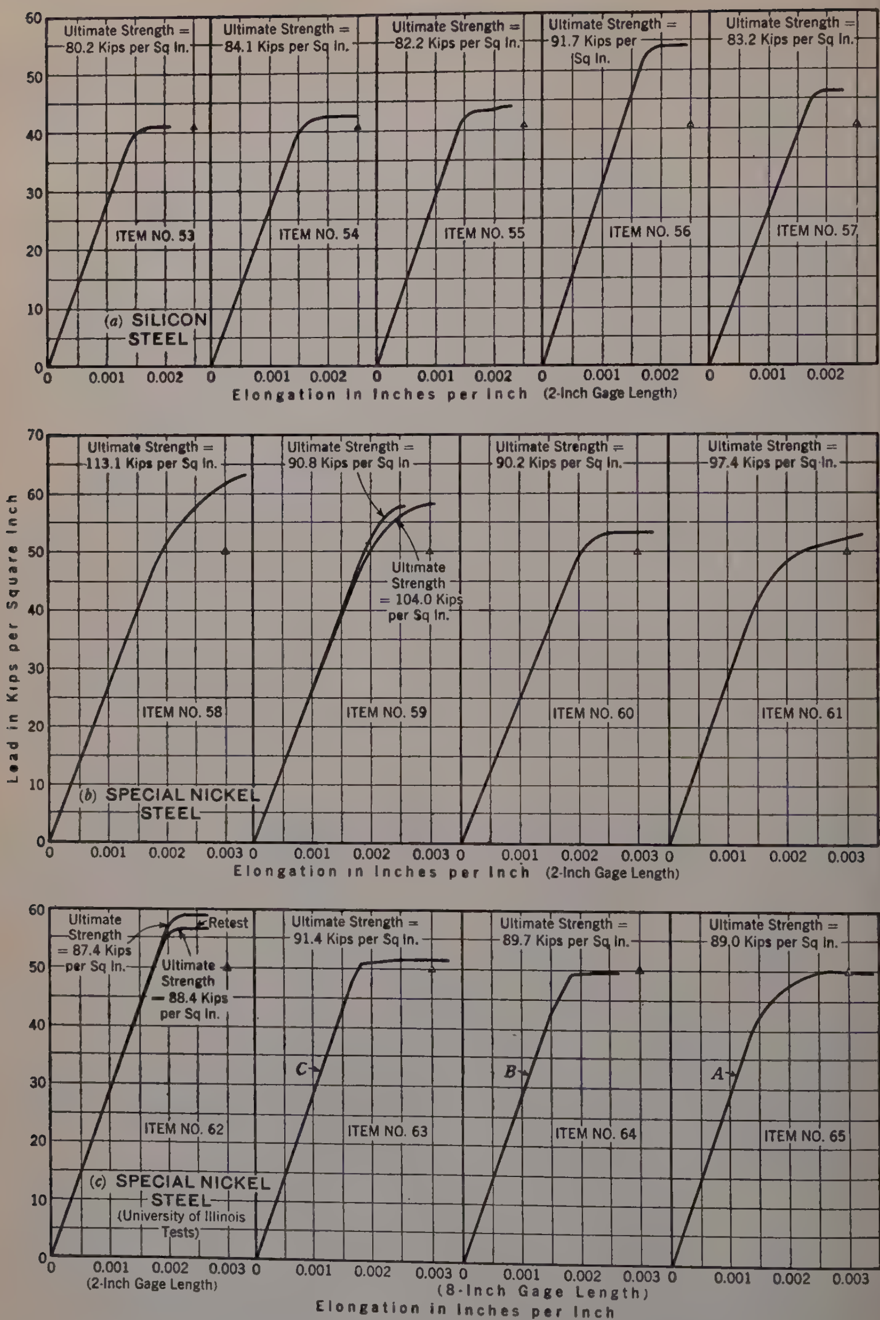
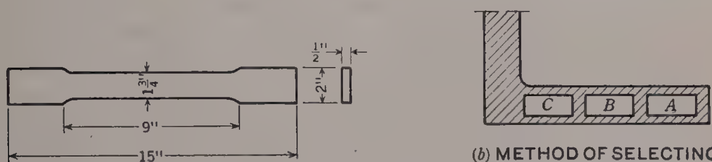


FIG. 34.—COMPARATIVE TESTS OF SILICON AND NICKEL STEEL COLUMNS, DELAWARE RIVER BRIDGE (SEE TABLE 15).

are satisfactory and will result in eye-bars that will meet full-sized test requirements:

	For high-tensile strength eye-bars
Percentage of carbon.......	0.30 to 0.40
Percentage of manganese....	0.50 to 0.75
Tensile strength, minimum, in kips per square inch....	70
Yield point, minimum......	0.5 of the actual tensile strength



(a) MACHINED SPECIMEN (b) METHOD OF SELECTING SPECIMENS

FIG. 35.—STANDARD TEST SPECIMENS, DELAWARE RIVER BRIDGE.

All specifications for chemical and physical properties of specimen tests for the special high-tensile grade steel must be referred to the manufacturer of the eye-bars. The modulus of elasticity of all grades of eye-bars is approximately 29 000 000 lb per sq in.

Brinell tests for hardness are sometimes required on heat-treated bars. They are made to determine the uniformity of hardness throughout the length of the bar. These tests are for information only, and do not form a basis of acceptance.

Method of Manufacture.—The process of making heat-treated eye-bars is, briefly, as follows: The head on one end of a bar is formed by repeating the operations of heating, upsetting, and rolling, until the correct proportions are attained. A hole is punched in the head while hot, somewhat smaller than the finished bored hole, and the bar is allowed to cool. After cooling, the bar is sheared to the correct length, turned around, and a head is formed on the opposite end of the bar in the same manner as described previously.

This process insures a quantity of metal adequate to form the head completely. Excess metal is squeezed out between the dies on the two sides of the head and, later, is sheared off while hot. The rough edges on the sides of the head due to this shearing, are chipped smooth.

Heat-treatment of high-tensile and special high-tensile eye-bars consists of heating, quenching, and tempering. After removal from the tempering furnace, the eye-bars are cooled in the air. The bars are straightened, and the pin-holes in the two heads are bored simultaneously. When Brinell tests are made, they are made just prior to boring.

Full-Sized Tests.—Orders for eye-bars customarily include a small percentage (about 3) of full-sized test bars which are tested to destruction in a 2 000-ton hydraulic horizontal testing machine. Test bars are selected from the finished bars except that those from bars too long for the testing machine are selected from the full-length bars after the heads on one end have been

formed. They are then cut, and the second head is formed to make the longest bars that can be tested.

The maximum distance, from center to center of pin-holes, with the ram of the testing machine at the beginning of the stroke, is 42 ft 6 in. The minimum is 3 ft 9 in. The length of stroke is 8 ft. The desired maximum length of the test bar is 35 ft.

Details of Eye-Bars.—High-tensile and special high-tensile heat-treated eye-bars are made in widths of 10 to 16 in. and in lengths, from center to center of pin-holes, of from 72 ft 7 in. (maximum) to 13 ft 0 in. (minimum). The excess thickness of head over body is about $\frac{1}{16}$ in. for Grade HT and $\frac{1}{8}$ in. for Grade HT Special. The minimum ratio of pin diameter to width of bar should be 0.875 for Grade HT, and 0.93 for Grade HT Special eye-bars. The pin clearance should be $\frac{1}{32}$ in. for Grade HT and $\frac{1}{50}$ in. for Grade HT Special. They can be made in sizes smaller than 10 in., but it is usually not economical to do so. Eye-bars more than 60 ft long are difficult to handle, and the cost of their manufacture is increased.

Conclusions

The use of special alloy steels has become increasingly prevalent, and is covering a wider range of structures as engineers are becoming more and more acquainted with its advantages.

Acknowledgments

The material which forms the basis of this paper was collated from a number of articles in *Engineering News-Record* and various papers published by the American Society of Civil Engineers, especially the Progress Report of Sub-Committee No. 2, Committee on Steel, of the Structural Division on Structural Alloy and Heat-Treated Steels[81]. The writer is also indebted to Leon S. Moisseiff, Jonathan Jones, and C. F. Goodrich, Members, Am. Soc. C. E., for information in their files.

[81] *Proceedings,* Am. Soc. C. E., March, 1936, p. 361.

EVOLUTION OF HIGH-STRENGTH STEELS USED IN STRUCTURAL ENGINEERING

By Leon S. Moisseiff[82], M. Am. Soc. C. E.

Synopsis

The development which led to the use of higher strength steels for bridges in the United States is outlined in this paper.

The rapid increase of population by natural growth and immigration and the industrialization of the country demanded transportation facilities for freight and men. Heavy trains required bridges of great strength, and crowded cities demanded structures with many tracks and wide roads. The wealth of the country and the growth of automotive transportation called for comfort and speed and led to the building of large bridges of long spans. Engineers, by the use of higher strength steel, were able to construct many great structures. The reasons governing the design of most large American bridges afford an excellent illustration of the evolution of the use of higher strength steels, and are related in detail.

The practical advantages of higher strength steels are feasibility of construction and economy of cost. They are readily seen. The limitations of these steels are not quite so apparent. They are discussed at length from their technical and practical aspects.

Introduction

During Man's long struggle for existence with the forces of Nature he has acquired a mass of knowledge about phenomena around him and a number of ways of manipulating and forming the many materials among which he found himself. Out of the want of Man grew his knowledge of Nature and his ability to appraise and apply the strength of materials. Under the incessant pressure of his surroundings he acquired tools and skill in handling them and rules and numbers for Nature's behavior. In the course of time he learned that by applying these rules the same task could be done with less labor, at the same time, creating a better product; and thus labor began to be co-ordinated, rules pertaining to natural phenomena grew into science, and the planning and execution of work became engineering.

In the course of this evolution, engineering became the application of scientific principles to meet the practical needs of Man in the best way, with the least expense of labor and materials. It is based on necessity and economy, which are two hard masters to please. At times, one or the other prevails, but necessity is the more compelling.

[82] Cons. Engr., New York, N. Y.

Transition from Wrought Iron to Steel

The meandering road of evolution is well illustrated by the changes that occurred during the transition from wrought iron to medium steel and from that to soft steel, in the United States and Central Europe.

Building Construction.—At about the beginning of the Twentieth Century steel came into use for frames and floors of multi-storied buildings. The growing demand for office buildings, hotels, and high-class apartment houses foretold the consumption of an enormous tonnage of steel for construction purposes. It became clear to the makers of steel that, although there would be a demand for large tonnage, there would also be a demand for a low-cost steel to compete with other building materials. It was realized that high strength and special qualities of steel were not required for this type of structure, which transmitted its loads directly to the foundations and which was not subject to impacts and reversals of stress.

It was important, from the point of view of the consumer, that the fabricated steel should be bought at a low price and that it be dependable; and from that of the producer, it was important that it should be manufactured in large tonnage without special precautions and manipulations and that it could be easily punched and fabricated. Above all, it was important that the fabricated product should be acceptable to the consumer and that there should be no rejections. This meant the avoidance of interruptions and the advantages of mass production. The demand, in the case of such relatively simple structures, was not for a steel of higher qualities developed like a race horse—highly nervous in behavior—to meet special requirements, but a dependable material that could stand abuse in fabrication—developed like a truck horse and possessing as little temperament. This was achieved by abandoning the medium steel that had prevailed before that period for bridges and adopting a soft steel of an average tensile strength of 60 kips per sq in., and a yield point, then known as "elastic limit," of one-half the tensile strength, as the standard material for engineering structures. To be more exact, the limitations for tensile strength were from 55 to 65 kips per sq in.

Steel Manufacture.—The demand at the beginning of the Twentieth Century for steel led to far-reaching re-organization of the steel manufacturing companies. A process of agglomeration set in; smaller mills were consolidated into large units, and in the process of co-ordination, many were abandoned in favor of enormous plants with capacity for large tonnage. Dispersed and competing fabricating companies were united in large groups which were more than able to compete with the remaining smaller independent organizations.

It cannot be regarded as a mere coincidence that while such re-organization took place in the United States, a similar process was effected at the same time in Central Europe, especially in Germany. It is still less accidental that, there also, after a systematic study of the advantages of soft steel, it was adopted as the standard material. By these changes and re-organizations the manufacturers were able to offer the market a uniform

and dependable steel at a reasonable cost and a wide confidence in the product was created. In fact, in rare instances only did the consumer exercise his right of inspection at the mills and shops; the material was accepted on the Manufacturers' Certificate. Time has shown that, in general, the development was in the right direction and that the confidence in the product was justified.

Transportation.—The rapid growth of the United States and the great development of its industries brought with it enormous demands for transportation facilities for freight and men. One has but to consider that during the fifty years from 1880 to 1930 more than 27 000 000 people immigrated into the United States and that, in the same period, the total population of the country increased from 50 000 000 to 123 000 000, to realize the tremendous impetus given to transportation. The revenue of the railroads increased 64% in the first decade of the century and in the first two decades it increased three and one-half times. The growth of commerce and industry produced a demand for transporting the increased tonnage of freight and the greater number of passengers created by the new conditions. This demand was met by the double-tracking of existing railroads and by the use of longer trains and more heavily loaded cars, which required heavier locomotives with greater axle loads. By about 1900 the heaviest axle loading specified for railroad bridges was 43 000 lb; the weight of one engine and tender was 145 tons and the load following it 4.2 kips per lin ft. The 1935 Specifications for Steel Railway Bridges of the American Railway Engineering Association call for an axle loading of 72 kips per lin ft, followed by a uniform load of 7.2 kips per lin ft. The weight of one locomotive and tender is 256 tons. The increase in axle load is two-thirds and that in the engine, three-fourths. The comparison of these specified figures illustrates vividly the growth in the live load of modern railroad bridges.

Bridge Construction.—The double-tracking of railroads brought with it the building of new bridges for heavier loads. At the same time, the increase in revenue-producing freight and passenger traffic made it financially worth while, in some instances, to extend railroad lines and to bridge wider rivers, and, in other instances, to shorten routes and reduce grades. All these effects of the growth of railroads called for stronger and longer bridges.

The larger number of railroad bridges of the period were built of the new standard soft steel of a tensile strength of 55 to 65 kips per sq in. It was then known as the "railroad bridge grade" of steel. Whenever bridge engineers desired a higher grade for their bridges (largely because they felt that the severe strains and impacts to which the bridges were subjected justified a stronger steel) the mills rolled a special grade of a tensile strength of 60 to 70 kips per sq in., and a yield point of one-half that of the ultimate strength. The proximity of the tolerance limits of this steel to the standard steel being 5 kips apart, permitted overlapping and gave the mills a chance to use the higher melts of the standard steel for the special steel.

Demand for High-Strength Steel

Some bridge engineers, however, were not satisfied with the two grades of steel offered by the manufacturers and insisted on a higher grade of material for their bridges. They specified a steel of a tensile strength of 62 to 72 kips per sq in., and a minimum yield point of 35 kips per sq in. The steel industry looked with scant favor on specifications which called for material different from the standard product. The demands of the engineers were not always readily met and their path was not "strewn with roses." In spite of discouragement, individual engineers adhered to their demands, and a number of larger bridges were built of this higher strength steel. Finally, it became quite usual for engineers to ask for a special carbon steel for more important bridges.

It is quite in order that a structural engineer should demand for his particular structure a material which physically may be better suited for its task, provided he can obtain it from the manufacturers at a price that will justify it economically. It is also in order that the manufacturers should not wish to disorganize their regular procedure of production, both in the mills and in the shops, unless the demand is great enough to compensate them for it. As the demand for the higher quality of material becomes larger and steadier the manufacturers finally see the necessity and the reasonableness of acceding to the demands of the consumers, and they realize the advantage of adopting the modified material as their standard.

Specifications.—More recent changes in the economy of the commercial and industrial life of the country and, no doubt to some extent, the competition of reinforced concrete, have led to the consideration of a slightly stronger steel which will enable engineers to produce their structures more economically. The tendency has finally crystallized in the adoption of a specification by the American Society for Testing Materials (Standards Serial A7-34), which forms the basis of the Specifications for Steel Railway Bridges of the American Railway Engineering Association.

These specifications are a product of the co-operation of the consumers and the producers, the designing engineers, and the mills. The specified material is practically the medium steel of 1903. Its physical qualifications are: Tensile strength, 60 to 72 kips per sq in., and yield point, one-half the ultimate strength, but not less than 33 kips per sq in.

Economy.—It is understood that the manufacture of this steel will cause no increase in cost to the mills. A slight increase in the cost of fabrication in the shops may be expected, due to the greater wear and tear on tools and machinery. The same material will probably be used for all kinds of structures and will thus become the standard American structural steel.

Rapid Mass Transportation.—While the foregoing developments in railroad transportation were progressing and exerting their effect on railroad bridges, similar and still more important changes were modifying life in cities and affecting the character of city bridges. The extraordinarily rapid growth of population in cities due to immigration and the inflow from

farm to city produced overcrowding and overflow into suburbs. The effect on the means of transportation is readily understood. The mechanism of commerce and industry is based on the co-ordination of many agencies, which can function properly only when the precision of time is observed. Many people must come to the city—to mills, shops, stores, and offices—and they must come there within a definite time on every working day. Moreover, the time is the same for each of them, within an hour or two. They also must return to their homes within other hours as closely set. Thus, the busy life of the city has produced the term, "rush hours." To transport this great and bustling mass of people, electric cars were running on busy streets and, later, entire trains were operated over elevated or subway structures, to achieve high-speed traffic uninterrupted by crossings. Larger and heavier cars were required and longer and longer trains were operated, attaining a length of ten cars in the Subway System of New York City. The higher speed of rapid-transit trains increased the danger of collisions and telescoping, and the underground operation of these trains made the adoption of all-metal, fire-proof cars imperative. All these requirements added weight to the cars. Thus, the weight of rapid-transit cars filled with passengers grew from 1 to 2 kips per lin ft of train.

In some cities the cars and trains had to be brought in over large streams requiring bridges with long spans. The bridges had to provide sufficient passage for cars and trains converging from many directions, and multiple tracks had to be provided on the bridges. At the same time, the increase in the wealth of the country justified the replacement of slow ferry traffic by long-span bridges. The growth of cities and the increased density of their population demanded provision for much heavier live loads than the old-time city bridge. It also demanded provision for the future growth of the frequency and continuity of these live loads.

Automotive Transportation.—During the twenty years, 1916 to 1936, a new factor in transportation began to exert a marked influence on the financing and building of bridges which, properly, should receive consideration. Favored by the increase of wealth due to the World War, the enormous development of automotive transportation of passengers, as well as of freight, brought with it new demands on bridges. Many capacious first-class highways were built throughout the United States and millions of automobiles are rolling over them at ever-increasing speeds. The new mode of travel by machine differs much from the old way by horse-drawn vehicle. It is not limited to immediate neighborhoods; its mileage is unbounded. It is swift and impatient; time has become essential. The automobilist, therefore, is willing to pay for his undelayed travel; and he does pay.

Long-Span Bridges

Toll Bridges.—The economic aspect of bridges thus took a new turn because of automobile travel. Although toll bridges were built during a hundred years, all over the country, they were able to pay on the investment only because the structures were relatively small and cheap to build, and the

low toll that the traffic could bear was sufficient to make the enterprise successful. Larger bridges could not be built on the expectations of income from tolls before the arrival of the automobile. The Brooklyn Bridge is a fair example. Gradually, the older toll bridges were being taken over by the State authorities and made free or replaced by modern bridges. All this was changed by automotive traffic. The unlimited traveling radius of the automobile and its high speed multiplied the number of vehicles on the highways and the higher standard of operating expenses made it possible and reasonable to charge substantial tolls.

The financial possibilities of building first-class bridges of longer spans were changed by the new factor completely. Where municipalities and Government agencies heretofore could build bridges only by borrowing funds, carrying and amortizing the bonds and maintaining the structures from general taxation, the possibility now offered itself to maintain the bridge, pay the interest and amortize the bonds within a reasonable number of years from the income collected by tolls. The opportunity was quickly seen and taken. Practically all long-span highway bridges built in recent times are toll bridges. All the large highway bridges have been financed on the basis of toll income: The Delaware River, the Ambassador, the bridges of the Port of New York Authority, the New Orleans, Tri-Borough, Golden Gate, and San Francisco-Oakland Bay Bridges, and numerous less costly structures all over the country.

With this development new problems in the economic relations of bridges appeared. It was not enough to build a safe, substantial bridge of good appearance to accommodate present and future traffic but, first, the structure had to be designed not to cost more than the traffic would bear, and, for privately built bridges, a fair profit had to be earned. The full import and weight of economic design and construction made itself felt in the planning of toll bridges.

Highway Bridges for Motor Traffic.—With the development of highway bridges for automotive traffic, new conditions were created. Although the live load per lane of traffic on a bridge is lighter for automotive travel than for rapid-transit trains or trolley lines, the dead load of the bridge is greater. Automotive travel requires a floor with a continuous non-skidding surface, preferably fire-proof. Heavy automobile trucks have high axle concentrations which must be sustained at any point of the roadway by the pavement and its supports. The dead load of highway bridges has increased as compared with the live load. This shift of loads exerts an influence on the selection of the type of the most suitable and economical structure. Suspension bridges display their best advantages with high uniform loads and, therefore, have come to the foreground. Heavy dead load and relatively light live load also favor higher strength steels.

Economy of Materials.—The heavier live loads and the longer spans of bridges placed before bridge engineers the problem of technical performance. The question arose whether the available engineering materials were adequate and efficient to sustain the great forces developed in large bridges. Necessity demanded a material stronger than the available structural steel.

It is an essential characteristic of a bridge that it has a tendency to use stronger materials in its construction. It spans a horizontal distance by transferring the gravitational forces on the span to its abutments. The magnitude of the resisting forces increases with the weights on the span and with their distance from the bridge abutments. It is evident that the longer the span the heavier will be the bridge and the greater will be the forces required to uphold the equilibrium. To resist these forces acting through the material of the bridge, either more material (and, therefore, still more weight) is required, or a material of higher strength must be substituted which will resist the greater forces without adding to the weight of the structure. This states in different words what is known to all engineers, that with an increase of span a bridge becomes heavier at a rate greater than in a simple proportion. Thus, in the longest highway bridges, the ratio of dead load to live load becomes 5 to 1 in the George Washington Bridge and 5.25 to 1 in the Golden Gate Bridge.

Heavy and long-span bridges, therefore, will require excessively large members to meet the enormous forces developed in them. The sectional areas and weights of the members become difficult to fabricate, to connect, and to handle. The structure appears to outgrow its proportions, which are intimately tied not only to the bridge as completed but also to the machinery and capacity of the fabricating plants and the available transportation facilities. Finally, the cost of the structure becomes excessive and defeats its object economically. The application of suitable and stronger steels frequently makes the building of the bridge feasible and financially possible.

The aim of all bridge engineers, therefore, has always been to reduce the weight of the longer bridges by utilizing materials of higher strength. The history of bridge engineering shows that almost in all instances of longer bridges their designers have endeavored to obtain a material which, either by its strength or its dependability, would enable them to allow higher intensities of stress, and thus reduce the weight to be carried. Generally, this has been accomplished with more or less success.

Reasons for Adopting Higher Strength Steels

So far the general outline of the play of social and economic forces which has led to the demand for stronger steel has been discussed in general without direct application to built structures. Generalizations afford a bird's-eye view and give a historical panorama, but they do not supply the direct information and practical understanding which concrete examples offer. These will be brought out in the following paragraphs.

In relating the various reasons which have led engineers to adopt a higher strength steel for the several structures which will be discussed, the writer wishes to call attention to an observed phenomenon in the records of human investigations and history. In the meandering of human thought and in the constant play of forces, interests, and changing conditions, events and decisions evolve neither along smooth straight lines nor along continuous curves. Many a jolt is experienced and frequently the chain of events is

interrupted. It is only by looking backward through a duration of time that people are able to perceive the general tendency and its flow and to describe it as sequences and logical action. Thus, the observer on the shore may see far away a smooth sea and a placidly moving ship.

Limitations to Progress.—The structural engineer in the past has had little choice in his search for the higher strength material. Laboratory tests, no matter how carefully made, can only indicate the quality of the product and establish its probable behavior when produced under mill conditions in large tonnage. Engineers have also learned, sometimes to their sorrow, that the mass product may develop failings and defects that cannot be anticipated in the laboratory samples, and even in small ingots. The engineer has often had to act in the rôle of stimulator and pioneer, and the manufacturer has brought out the new material with more or less enthusiasm and apprehension. It involves a risk in money and reputation to contract under specifications, which necessarily are rigid, to roll and fabricate the new steel. A higher price must necessarily be set to cover the cost of uncertainties and novelty to equipment and personnel.

Several instances may well illustrate the reasons which prompted the introduction of the use of nickel steel for bridges. At the beginning of the century two bridges over the East River—the Queensboro and the Manhattan—were being planned by the Department of Bridges of New York City.

Eye-Bars in the Queensboro Bridge.—The Queensboro Bridge is a double cantilever structure with two main spans of 984 and 1 182 ft. Originally, it was designed for two rapid-transit tracks, four trolley tracks, a roadway of 35.5 ft, and two sidewalks; after the letting of the contract two more rapid-transit tracks were added. In accordance with the loads then established, the specified congested live load was 16 kips per lin ft, and the average dead load, 35 kips per lin ft. The dead and live load stresses in the panels of the upper chord attain a maximum stress of 19 000 kips. Using the proportional unit stress which could be allowed on eye-bars of medium steel (20 kips per sq in.) about 950 sq in. of bars would be required. As built, the nickel-steel eye-bars have a greatest sectional area of 680 sq in., consisting of 20 bars, 16 by $2\frac{1}{8}$ in. An increase of 40% in sectional area would thus be required if the bars were made of carbon steel. As the largest eye-bars available were used, it meant a proportional increase in the number of bars. The increased weight of the tension members would add to the dead load and, in its turn, would require more bars. Structurally, it meant that the width of the chord and the length of the pins would have to be increased, which again would lead to further increase in dead load.

An increase of 40% in the number of bars would have required the widening of the upper chord in the panels of maximum stress from its present width of 90 in. to more than 120 in. The greater number of bars in the chord, in turn, would involve the widening of truss members to unwieldy proportions.

The engineers discussed various alloy steels with metallurgists and manufacturers, and, finally, the Carnegie Steel Company, in 1902, made a heat of steel containing 3.26% nickel, 0.21% carbon, and 0.53% manganese. From

it 21 eye-bars were manufactured by the American Bridge Company in sizes of 6 by 1 in., 6 by 2 in., 8 by 1½ in., and 10 by 1¼ in. Tests were made of specimens and of full-sized eye-bars, annealed and not annealed. The results of the tests indicated that a good structural material had been obtained which could be depended upon for bridge purposes and which, after further experimentation, could be specified to meet a yield point of 48 to 50 kips per sq in. and an ultimate strength of 100 kips per sq in.

In 1903 a further set of eight bars of basic open-hearth steel, containing 3.24% nickel, was made and tested. When tested, the full-sized eye-bars developed an elastic limit of 46 to 49 kips per sq. in., and an ultimate strength of 84 to 87 kips per sq. in. This gave assurance that a high-strength steel could be manufactured into eye-bars on which, for dead load and regular live load (8 kips per lin ft), a tensile unit stress of 30 kips per sq in., and for dead load and congested live load (16 kips per lin ft), a stress of 39 kips per sq in., could be allowed. During the construction of the bridge the bars were ultimately made to meet a minimum elastic limit of 48 kips per sq in., in full-sized tests, and an ultimate strength of 80 to 85 kips per sq in.; elongation in 18 ft, 9%; reduction of area, 40%; and bending, 180° around a pin of twice the thickness[33]. In this first application of nickel steel to bridge construction about 6 000 tons of eye-bars and pins were consumed.

The introduction of nickel as an alloy to the steel for the eye-bars of the Queensboro Bridge offers a good illustration of the causes which have led to the use of high-strength steels for bridges. A stronger steel was desired to hold the number of eye-bars and their section within manageable limits. The steel had to be as dependable and as workable as the standard bridge steel. A percentage of about 3.25 of nickel added to the steel supplied the necessary strength and quality, and this material was adopted for the eye-bars of the bridge. The type and general dimensions of the structure were not affected by the selection of the stronger steel.

Stiffening Trusses in the Manhattan Bridge.—Another illustration, but of a different character, of the use of high-strength steel is presented by the events which led to the use of nickel steel for the stiffening trusses of the Manhattan Bridge in New York City. It was planned at the same time as the Queensboro Bridge and was designed for four rapid-transit tracks, four trolley tracks, a 35.5-ft roadway, and two footwalks. For this capacity a congested live load of 16 kips per lin ft was adopted.

The dimensions of the bridge and its dead and live load created a problem for the engineers. The bridge had to clear the East River between pier lines and the length of the main span was determined by the location of the bridge and the width of the river between pier-head lines at 1 470 ft. A desire to present a well-balanced appearance made the side spans 725 ft. Before the development of the deflection theory of suspension bridges a reasonable design of stiffening trusses for a bridge with suspended side spans

[33] "Nickel Steel Eye-Bars for Blackwells' Island Bridge" (later, Queensboro Bridge), by William R. Webster, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXIV (1909), p. 289.

of similar dimensions, and a live load as heavy as two-thirds that of the dead load, would require trusses of great depth and extraordinary chord sections to come within allowable unit stresses and limited deflections as then computed. The introduction of the deflection theory led to a more correct determination of deflections and made it possible to design the bridge with relatively shallow trusses. Their depth of 24 ft was determined by the cross-section of the bridge, which has a double deck, and the required clearance for the rapid transit trains on the lower deck. It may be pointed out that this was the first suspension bridge of its kind and character designed in accordance with the deflection theory and built with shallow trusses.

The behavior of a suspension bridge is such that, given the general dimensions and the dead and live loads, its stiffness will be controlled more by these factors than by the stiffness of the suspended truss itself. This is explained by the fact that in the computations of the stiffening truss, the moment of inertia is not directly proportional to the deflection, and exerts a relatively small influence. However, it must follow such deflections as the system of forces on the cables and the elasticity of the latter will compel. In other words, the truss must bend in conformity with the cable deflections and it can only mitigate them. It is well known that in a beam of given depth deflecting to a given curve the stresses in the chords will be proportional to the degree of curvature. With the general dimensions of the Manhattan Bridge and its dead load a congested live load of 16 kips per lin ft will develop stresses of such intensity that the standard medium structural steel could not sustain them within its elastic limit. True enough, a uniform congested load moving on many lanes, either in unbroken or broken lengths, will rarely assume the positions which will cause maximum stresses in suspension bridges with side spans. In accordance with this argument the designers have given full consideration to the probability of maximum stresses in the stiffening trusses and have allowed high unit stresses on the selected material. The adopted material, however, had to have an elastic limit high enough at no time to cripple the trusses by exceeding that limit.

After the results obtained from the tests of nickel-steel eye-bars for the Queensboro Bridge, it was a matter of course for the engineers to turn to nickel steel as a material which can develop a sufficiently high elastic limit and ultimate strength for the moments and deflections of the stiffening trusses. Accordingly, the specifications for the nickel steel for the stiffening trusses of the Manhattan Bridge called for the following physical requirements:

Ultimate strength	85 to 95 kips per sq in.
Elastic limit, minimum........	55 kips per sq in.
Percentage of elongation in 8 in.	$\frac{1\,600 \text{ (minimum)}}{\text{Ultimate strength}}$
Percentage of reduction of area.	40 (minimum)

As shown by the results of the acceptance tests of all the heats rolled, the material furnished actually developed an average elastic limit of 55.9

to 59.8 kips per sq in., and an average ultimate strength of 87.2 to 90.8 kips per sq in. Having in mind the high intensity of loading assumed and the low probability of the positions of maximum stresses, a stress of 40 kips per sq in. in tension, properly reduced for compression, was allowed on this material. The permissible unit stress was thus 73% of the elastic limit of the steel. Altogether 8 000 tons of nickel steel were consumed in the Manhattan Bridge.

The recently proposed revised A. S. T. M. Specifications for Nickel Steel differ from those of the Manhattan Bridge in 1906 only by raising the tensile strength to 90 to 115 kips per sq in. and by lowering the reduction of area to 30 per cent.

In the case of the Manhattan Bridge the introduction of a stronger steel was due to a necessity which came from the character of the type of the structure and its dimensions and loads. If there were no steels of higher elastic limit available the bridge could not have been designed as it was. In this case the type and general dimensions of the structure were substantially affected by the selection of the stronger steel.

Nickel Steel in the Quebec Bridge.—In the construction of the present Quebec Bridge the Board of Engineers, which was created after the failure of the first bridge during its erection, make painstaking and careful studies of all phases of the work. It made extensive tests of carbon and nickel steel eye-bars and columns and, after careful considerations, it incorporated 16 000 tons of nickel steel in the structure. In the report[34] on the Quebec Bridge, the Board states: "Nickel steel was used wherever it would effect a saving in weight of the structure as a whole, and, by this means, notwithstanding its higher price, be a factor in reducing total cost."

Nickel Steel in the Delaware River Bridge.—The Delaware River Suspension Bridge, between Philadelphia, Pa., and Camden, N. J., was designed for a congested live load of 12 kips per lin ft. It has a span of 1 750 ft and a reinforced concrete roadway 57 ft wide; its design dead load is 26 kips per lin ft. The same considerations that led to the adoption of nickel steel for the stiffening trusses of the Manhattan Bridge led to the use of a high-strength steel for the trusses of the Delaware River Bridge. Realizing the need of high-strength steels in bridge building, the engineers of the Delaware River Bridge had in mind the possibility of opening the field for such steels, in order to encourage steel makers to produce various suitable high-strength steels and to foster competition. The specifications for the stiffening trusses, therefore, called for a special steel having a minimum yield point of 55 kips per sq in. and a minimum tensile strength of 90 kips per sq in., without specifying a definite alloy. It gave a free opportunity to steel manufacturers to offer an alloy to meet the physical requirements. The manufacturers finally furnished a nickel steel containing an average of 3.2% nickel.

One of the peculiarities of stiffening trusses in suspension bridges is that the shear is relatively low and that the web members are relatively light

[34] "The Quebec Bridge": Report of the Govt. Board of Engrs., 1919.

It was found that the web members in the trusses of the Delaware River Bridge, if made of nickel steel, would under-run the desirable minimum sections, and, consequently, they were made of silicon steel. This is the first instance in which a differentiation was made in members of a truss by using higher and lower steels. A total tonnage of 5 150 tons of nickel steel was consumed for the bridge.

The San Francisco-Oakland Bay Bridge.—A still more recent illustration of the use of nickel steel is furnished by the San Francisco-Oakland Bay Bridge in which the high-strength steel was used in two structures of the same bridge for different reasons. This project consists of a twin suspension bridge over the West Bay and of a cantilever structure and nineteen truss spans over the East Bay. It is a double-deck structure. The main span of the cantilever structure is 1 400 ft and the especially heavy dead load, due to the wide roadways, made it of major importance to hold the size and weight of the truss members within reasonable limits. To minimize the sizes of the stiff members 3 400 tons of nickel steel were consumed. The reasons for using stronger steel here were purely technical and not economical.

Still another instance in which a higher strength steel was used for technical reasons is furnished by the center anchorage of the suspension structure. The West Bay is about 10 000 ft wide and is crossed by twin suspension bridges, each having a main span of 2 310 ft. For a uniform fixed load over the entire length, the structure could have been built with three main spans and two side spans; but for a moving live load of 7 kips per lin ft, the bridge would be too flexible both for the towers and for the stiffening trusses, and would thus become impassible for traffic. To limit the deflections of the bridge a central anchorage pier was built which will restrain the motion of the cables by the stability of its weight. A simple solution was found to connect the cables to the pier and to each other by positive means. The cables of each of the bridges which meet at the central anchorage are displayed from their compacted cylindrical shape the same as at the land anchorages. The individual strands that constitute the cables are laid over thirty-seven anchor shoes, each of which is engaged by a pair of eye-bars similar to the usual suspension-bridge construction. Here, the similarity ceases. The eye-bars are not connected to other bars embedded in concrete, but at their free end they are connected by pins to vertical steel plates. In this way the plates are only intermediate links connecting the cable ends of the twin bridges. The cables are thus tied to each other continuously by bars and plates of steel. The steel tie-plates are made large enough (23 by 35 ft) to extend downward to provide for a pin connection to a steel structure anchored and embedded in the masonry of the central anchorage pier. The anchorage restrains the horizontal motion of the cables. When it is considered that the pull in the cables at the central anchorage may reach 36 400 kips per cable and that the unbalanced pull may attain 10 600 kips, the importance of the tie-plates—their size and their function—can be realized. Each cable connects to four plates by

means of thirty-seven pins and the plates connect to the anchorage by a pin 27 in. in diameter. The geometrical dimensions are such that the distribution of stress in the plates is of the greatest importance. Although, in accordance with theoretical computations, the strength of silicon steel plates was sufficient for the purpose, the engineers felt that excess of strength in the steel was of special value in this case and that the well-known toughness of nickel steel would be much in place. The tie-plates, therefore, were made of nickel steel, requiring about 280 tons. In this instance, merely technical considerations and not economy were the determining factors. Not only was the higher tensile strength of the material considered but also its toughness.

Manganese Steel in the Bayonne Bridge.—In 1927 the engineers of the Port of New York Authority were planning a bridge over the Kill van Kull connecting Staten Island, New York, with the mainland at Bayonne, N. J. After much study it was decided to build an arch with a single span of 1 675 ft between centers of bearings. The trusses were designed to act as three-hinged arches with the lower chords as continuous ribs from end hinge to end hinge. The lower chord was designed to sustain all the dead load and part of the live load, the upper chord and the web members acting merely as a participating and stiffening truss.

The greatest reaction on the pins is nearly 31 400 kips per truss. It was desirable to limit the thickness of the assembled plates and splices for riveting purposes but, above all, considerations of weight were governing. For every pound of weight added an additional pound of steel was required to sustain it; economy, therefore, decided the issue.

At that time, advanced engineering had already recognized the fact that the function of the various parts of a structure and that of their constituent members and details, demand in some parts greater strength and in others greater stiffness. It had realized that it is good design to utilize various grades of available steel and that, by the proper use of these various steels, a better and a more economical structure can be built. Modern design, therefore, will utilize steel of various strengths. Where the usual stresses must be met and where stiffness is of importance, the common carbon steel will be used; where greater forces must be sustained, medium strength steel such as silicon will be generally utilized. Finally, where exceptionally great forces are met and the members become excessively bulky, high-strength alloy steels will be introduced. This tendency was recognized systematically in the design and the proportioning of the various parts and members of the Bayonne Arch. The main material of the lower chord is high-strength alloy steel throughout; that of the top chord is silicon steel, except the three end panels which are of carbon steel; all the web members are of carbon steel, except some which were made of silicon steel because of erection stresses; all lateral bracing and sway-bracing are of carbon steel.

For the high-strength steel an alloy containing 3.25% nickel and having a minimum yield point of 55 kips per sq in. and a tensile strength of

90 kips per sq in., was specified. In the specifications it was stated, however, that, with the approval of the engineer, the contractor might substitute for the nickel steel another steel which would meet the physical requirements for nickel steel and in all other respects be equivalent in quality. The American Bridge Company, as the lowest bidder, was awarded the contract, and it offered as an equivalent substitute a carbon manganese steel at a reduction of $15 per ton below the price of nickel steel. The carbon manganese was to have the chemical and physical qualities indicated in Table 22.

TABLE 22.—CHARACTERISTICS OF MANGANESE STEEL, BAYONNE BRIDGE

Description	Specification, Port of New York Authority	Tests of actual product§
Chemical Properties (Percentages):		
Carbon	0.40*	0.33
Manganese	1.80†	1.63
Phosphorus	0.04†	....
Sulfur	0.05†	....
Silicon	0.10 to 0.30†	0.18
Physical Properties:		
Yield point stress, in kips per square inch	55‡	58.6
Tensile strength, in kips per square inch	90‡	101.6
Percentage elongation in 8 in	1 600‡	19.5
	Actual tensile strength	
Percentage reduction of area	30‡	42.6

* Maximum allowable; preferably not more than 0.35 per cent. † Maximum allowable. ‡ Minimum allowable. § 168 melts and 968 tests.

After a careful investigation, which included tests from a heat especially rolled for the purpose, and the adoption of precautionary provisions and more rigorous requirements regarding testing and inspection, the Port of New York Authority permitted the use of manganese steel in substitution for nickel steel. The total saving in price was about $80 000.

The average results obtained on the standard tests of manganese steel rolled for the chords of the Bayonne Arch are cited for comparison in Table 22. The elastic limit, being the Johnston point, was found to be 46 kips per sq in. None of the steel specimens was heat-treated, and the steel was used in the "as-rolled" condition.

A series of tests of large-sized columns was conducted by the Port of New York Authority jointly with the National Bureau of Standards; among them were two of the type of the George Washington Bridge and two of the type of the Bayonne Arch. The strongest column, of carbon manganese, developed an ultimate resistance of 9 653 kips. The ultimate compressive strength of the carbon manganese columns was from 58.6 to 62.3 kips per sq in.[85] Thus, the column strength was found to be the average yield-point strength of the material in the members. It should be added that carbon manganese steel rivets were used successfully for the field connections.

[85] "Tests of Steel Tower Columns for the George Washington Bridge", by Stang and Whittemore, *Journal of Research*, National Bureau of Standards, Vol. 15, September, 1935, and paper in print.

The manufacture of manganese steel has shown that much care must be exercised in the production of the steel and the rolling of the plates and that the heavier plates are likely to encounter difficulties and to develop cracks. Tension tests of large specimens made by the contractor in connection with the Bayonne Arch gave poor results, and he finally decided not to use carbon manganese steel in the tension members of that bridge. In this instance, the reasons for using high-strength steel was due, as stated, to economical considerations. Substitution of carbon manganese steel for nickel steel was due to a saving in cost to the consumer. It is doubtful whether there was an actual saving in cost to the producer.

Summary.—The writer has discussed at length the application of high-strength steel to most of the large bridges in the United States and the reasons which prompted its use. The reason that nickel steel was used in most cases was simply that it was the material available and had been tried out first. Manufacturers were familiar with it and engineers knew it to be dependable and well behaving in finished structures. The reasons that prompted metallurgists and engineers to search for other high-strength steels suitable for structural purposes were those of economy. When used in the Queensboro and Manhattan Bridges in the first decade of the Twentieth Century, nickel steel cost about 3 cents per lb more than carbon steel. This differential in price has varied but little during the succeeding years. For the Delaware River Bridge, in 1924, the average of the bids was 2.25 cents per lb more for nickel steel than for carbon steel; for the Bayonne Arch, in 1928, it was 2.8 cents; and in the San Francisco-Oakland Bay Bridge, in 1933, it was 2.8 cents. The alloying nickel is practically controlled by a monopoly, and its cost to the mills has varied little. Additional cost of rolling and fabricating has kept the price relatively high. For the Harahan Bridge, a cantilever railroad bridge over the Mississippi River, at Memphis, Tenn., in 1914, 700 tons of Mayari steel was used in eye-bars and riveted members, with the evident intent to reduce the cost of high-strength steel and, apparently, good technical results were obtained. Mayari steel, however, has not been in the market for structural purposes in recent years.

History of Silicon Steel

Fortunately for the advance of bridge construction, a new steel was developed about 1916 which took its place in strength above structural carbon steel and below nickel or special steels. This steel, known as silicon steel, has a minimum yield point of 45 kips per sq in. and a tensile strength of 80 to 95 kips per sq in.

Notwithstanding its great popularity, silicon steel is fairly young. More than fifty years ago a silicon steel was proposed by English mills for ship construction. However, not before 1907 did silicon steel make its appearance in ship construction in plates for the S. S. *Mauretania.* Because of the wide use of this steel in recent times, the writer considers it important to discuss what is generally associated with the trade mark, "silicon steel,"

in European countries. The steel used in the *Mauretania* is said to have had the characteristics cited in Table 23.

TABLE 23.—Evolution of Silicon Steel

Description	Silicon steel in the S. S. *Mauretania*, in 1907	Silicon steel in George Washington Bridge, in 1928–30
Chemical Analysis (Percentages):		
Carbon	0.27	0.35
Silicon	1.12	0.27
Manganese	0.72	0.78
Physical Properties:		
Yield-point stress, in kips per square inch	65	50.8
Tensile strength, in kips per square inch	92 to 105	88.8
Percentage elongation in 8 in.	25 to 30	22
Percentage reduction of area	......	43

In later years, European manufacturers, and especially German mills, after the World War, had adopted a similar grade of steel, usually containing from 1.0 to 1.3% of silicon. This steel gave unsatisfactory results and ultimately was recalled. The greatest trouble was the high percentage of silicon as compared with the practice of American mills which limits it to 0.45 per cent. Because of the troubles experienced with the higher percentage, silicon steels in general acquired a bad reputation in Europe. In discussing steels with European engineers it will be well to explain to them the difference between European and American steel of the same name. As manufactured in the United States, silicon steel has a low percentage of silicon and a medium percentage of manganese. It could properly be named "Mansil" to indicate the ingredients which produce its characteristic qualities, or "Medium Manganese" steel.

It first made its appearance in bridge building in 1915 in the bridge across the Ohio River, at Metropolis, Ill. This bridge has a simple truss span of 720 ft and its long span and weight demanded a higher-strength steel. The demand was well met by silicon steel. So successful was the new material that to date about 350 000 tons of it has been consumed for bridges. It has been used for the towers of the Delaware River Bridge, Ambassador Bridge, Martinez-Benicia Bridge, George Washington Bridge, Golden Gate, New Orleans, Tri-Borough, and San Francisco-Oakland Bay Bridges. It has become a standard steel of to-day.

A fair idea of the quality of silicon steel that is being manufactured in such large tonnage can be had from the results of 895 melts and 1 884 tests of the silicon steel used in the towers of the George Washington Bridge, a total of 23 600 tons (see Table 23).

Economy.—The reason for the extensive use of silicon steel is its economy. Erected, in place, it costs 10 to 15% more than carbon steel. Its specified minimum yield point is 36% higher and a permissible unit stress of one-third higher is generally allowed for it. An elementary example will illus-

trate the economic relation of silicon steel to carbon steel. Assuming that 1 000 tons of carbon steel, in place, will cost $100 000, the higher allowable unit stress, with practical allowances, will require only 800 tons of silicon steel at, say, $112.50 per ton, amounting to $90 000. A saving of $10 000, or of 10% of the cost, will be realized.

The saving in cost shown in the foregoing example covers the lesser cost of the material in place. It does not take into account the saving in steel effected by virtue of its decreased total weight. Although it is of slight importance in buildings and minor bridges, the reduction in weight becomes a substantial factor in longer bridges where an additional pound of weight may require 3 lb of steel to sustain it. To this factor is due the saving sought by bridge engineers, and it is the determining cause for the selection of higher strength steel.

Practical Advantages and Limitations of High-Strength Steel

From the point of view of the engineering of bridges and structures, a high-strength steel is an alloy that develops a substantially higher yield point and tensile strength than the so-called standard structural steel and is sufficiently ductile to be fabricated by the equipment and procedure in modern shops. It must also be ductile enough to overcome stress concentrations greater than the yield-point stress when they occur and to be capable of distributing the stress more or less over the entire section of the member without impairing its efficiency. The yield point is accepted as one of the measures of the quality of steel, not because it is a better index of quality than the limit of proportionality or the elastic limit, but simply because, for acceptance testing, it is much more feasible. Testing for commercial acceptance of material requires simplicity and speed. Both are obtained by the observation of the yield point or a conventional definition of it. The consumer, therefore, really buys the material on the basis of its yield point and its tensile strength as observed by testing procedures which are established conventionally to meet the exigencies of practical acceptance at the mills and shops. When the engineer specifies a high-strength steel he obtains a product with a higher yield point and higher tensile strength than is ordinarily used.

Criterion for Unit Stresses.—The object of the engineer in introducing a higher strength steel, whether for technical or for economical reasons, is to utilize its strength by applying to it higher permissible unit stresses. The question then arises here as to how much stronger is the new steel when in action in the structure than the common carbon steel. Can the "higher strength" of the new material be utilized by the designer of the structure in direct proportion to its yield point or tensile strength? If so, which of the two should guide the adoption of the unit stress, the yield point or the tensile strength? The engineer thus arrives at the fundamental considerations of what constitutes the resistance of steel when acting in a structure.

Engineering knowledge of the present time has found experimentally that a steel specimen subjected to a static load within its limit of proportionality

will sustain the load indefinitely and when the load is released it will assume its original dimensions. Very little set and scarcely any creep due to time has been observed on specimens. The good behavior and long life of innumerable steel structures bear witness to these facts. Sufficient observations are available, furthermore, to justify the assumption that the variation of the intensity of a load applied in one direction only will not reduce the sustaining qualities of the material. Experiments have not yet established positively that the tensile strength of a specimen will not be reduced by applying a load and then removing it entirely a sufficient number of times to establish faith in the probability that such conditions would not be exceeded during the life of the structure. It has been established, however, that a reversal of stress within certain limits will cause the specimen to fail after many repetitions. The strength of the steel to resist an "infinite" number of stress reversals is known as fatigue strength and is lower than its static strength. Moreover, the fatigue strength depends on the tensile strength and not on the proportional limit or on the yield point. Taking the lowest values commonly set in specifications, the percentages of increase in yield point and tensile strength are indicated by reference to Table 24.

TABLE 24.—PERCENTAGE INCREASE IN YIELD POINT AND TENSILE STRENGTH

Description	Yield point stress	Tensile strength
Carbon steel to silicon steel	36	33
Carbon steel to special steel	67	50

Although the decrease in percentage of tensile strength is not much for the two varieties of steel in Table 24, attention is called to the fact because several steels recently have come into the market, the main characteristic of which is a higher ratio of yield point to tensile strength. When establishing the permissible unit stresses for the design of the structure, therefore, the engineer must consider the character of the loads to which it will be subjected and must choose the material best fitted for its function. For practically static loads the higher yield point will be the controlling influence, and for frequently reversed stresses the tensile strength will be the controlling influence.

While establishing permissible unit stresses the engineer should keep in mind that the proportional limit and the elastic limit are the limits that should not be exceeded in a structure and that the true margin of safety depends on them. Careful observations have shown that the elastic limit is lower than the yield point and sometimes substantially so. Generally, the elastic limit can be taken at eight-tenths of the yield-point stress and it may be substantially below it.

With the foregoing limitations in mind the engineer can take advantage of the higher strength qualities of the special steels. The recent Specifications for Steel Railway Bridges of the American Railway Engineering

Association call for the following permissible unit stresses in tension (in kips per square inch, net):

Carbon steel	18
Silicon steel	24
Nickel steel	30

It will be noted that the stress allowed for nickel steel is 1.67 times that of carbon steel, and that allowed for silicon steel is 1.33 times that of carbon steel. As a result, smaller sectional areas are required which will require less material. The reduced weight of steel required means less dead load for the structure to sustain and, in turn, reduces the steel tonnage. Where sections of carbon steel are too large, they will become proportionally smaller for the stronger steels and members will become easier to handle. Above all, when properly and judiciously applied, a saving in cost of the structure will be realized.

Limitations Imposed by Flexibility.—There are limitations, however, to the use of higher strength steels for structures which are inherent in all steels and are derived from their basic qualities. The modulus of elasticity of various steels is practically the same within narrow limits. All phenomena of the behavior of steel members and structures which are functions of the modulus of elasticity, therefore, will develop the same degree of deformation. The members of the structure composed of high-strength steel will shorten and elongate in inverse proportion to their sectional area, and the area being smaller, they will develop proportionately greater deformation. The structure as a whole, consisting of all the members, therefore, will be less stiff and will deflect more under load. This is not desirable. The more flexible a structure is the more secondary stresses it will develop in its stiff connections and if the deformation of the structure should become appreciable it will also develop additional stresses due to the distortion of the original geometrical pattern. Thus, very tall structures when exposed to high winds may sway sufficiently to produce additional stresses to an appreciable degree, so also may tall towers of long-span suspension bridges.

Limitations Imposed by Vibration.—Where the rapid application of loads produces vibrations, the intensity of these vibrations will be greater for the stronger steels because the intensity is inversely proportional to the moment of inertia of the structure and the latter value will be smaller for the stronger steel. Trains passing over railroad bridges at high speeds cause much vibration in the structure, and it is of serious importance rather to reduce than to augment these vibrations. Both the greater deformations and the higher vibrations which will be produced in structures built of stronger steels demand the careful thought of the designing engineer. In designing a structure it is important to consider seriously whether the advantage of the reduced cost effected by the use of the stronger steel is not outweighed by the disadvantage of the more flexible structure.

Buckling of Plates and Flanges.—Another phenomenon in the behavior of steel members in which the constancy of the modulus of elasticity plays

an important rôle is that of the buckling of plates and outstanding flanges of steel members in compression. According to Euler's well-known theoretical study of the buckling strength of a deflected column under a compressive load, the column strength is directly proportional to the modulus of elasticity of the material and the moment of inertia of the column. The correctness of the Euler theory has been verified experimentally. The elastic stability or buckling strength of plates subjected to compression and shear has been derived from Euler's theory, and it has been established theoretically and verified experimentally that the critical unit stress at which buckling is imminent is directly proportional to the modulus of elasticity. A concise statement of the practical aspects of the problem has been presented [86] very ably by Otis E. Hovey, M. Am. Soc. C. E., who establishes a table of recommended ratios of the width of a plate to its thickness based on the yield point of the material used. For the purpose of showing that smaller ratios are permissible for higher yield points, three ratios will be quoted:

Yield point stress, in kips per square inch	Ratio of width to thickness
33	48.8
45	41.8
55	37.8

The foregoing data demonstrate vividly that a stronger steel is not proportionately more stable against buckling than common carbon steel and that when high-strength steel is applied it is sometimes penalized where wide plates are used in the structure.

The practical importance of the limitation presented by buckling resistance in design is well illustrated by the case of the towers of the Golden Gate Bridge and the San Francisco-Oakland Bridge. The towers of the Golden Gate Bridge are 690 ft above the piers and each pier must sustain a vertical load of 61 500 tons. About 45 000 tons of carbon and silicon steel were used for both towers. The tower shafts are built as cellular structures with widths of 42 in. The general dimensions of the towers are such that in the lower portion the stability of the steel plates, and not high stresses, is controlling. Carbon steel, therefore, was used in the lower two-thirds of the shaft. In the upper one-third where the increased slimness of the shaft requires higher allowable working stresses and stronger steel, silicon was used. It was found that a number of silicon steel plates, located toward the outer faces, required thickening because the buckling resistance was not equivalent to the permissible unit stress. A number of single web-plates, therefore, were increased in thickness; a total of about 70 tons of steel was thus added.

Similar increases in thickness because of buckling strength were made in some web-plates in the towers of the San Francisco-Oakland Bay Bridge.

Details.—In the building of a steel member many auxiliary parts enter which are commonly known as "details." Designing engineers know that

[86] "Elastic Stability of Plates", by Otis E. Hovey, *Bulletin*, Am. Ry. Eng. Assoc., February, 1935.

such auxiliary parts as lacing-bars, batten-plates, and stiffeners are not highly stressed and do not demand high-strength steel. In order to function well and to fulfill their purpose these parts should be rather stiff than strong. In other words, lacing-bars, batten-plates, and stiffeners should offer high resistance to deformation. To do this it is important that they have more section rather than that they be stronger per unit of area. These parts of high-strength members could very well be made of common carbon steel when they do not carry primary stresses and where there is enough tonnage to warrant the additional care required in sorting in the shop.

Similar considerations hold true for most secondary members in bridge trusses. Hangers and parts in subdivided panels sustain concentrated panel loads only and, as a rule, they do not require heavy sections. Lighter members frequently under-run in size and should be made of carbon steel. The same is true for most lateral and sway-bracing in medium span bridges. Sway-bracing mostly functions to preserve the geometrical pattern of the structure and stiffness more than strength, is of importance. Lateral bracing requires heavy members where the lateral shear becomes high, which generally is in less than one-half the length of the bridge; in the lighter half, the least size of the member controls. If the tonnage of the steel is sufficient to warrant the use of two kinds of steel, carbon steel may be used in at least a part of the lateral system.

Adapting Steels to Parts of a Structure.—In the design of large structures engineers have recognized the value of differentiating between various members according to their functions and of using steels suitable for these functions. For example, the web members and all lateral and sway-bracing members of the Bayonne Arch are of carbon steel, whereas the lower chord is of carbon manganese steel and the top chord of silicon steel. The towers of the Delaware River Bridge are of silicon steel and their transverse bracing, as well as all diaphragms, etc., are of carbon steel; the chords of the stiffening trusses are of nickel steel; the webs, however, are of silicon steel and the lateral system, because of participation stresses, is also of silicon steel. In the towers of the George Washington Bridge the shafts are of silicon steel, the bracing, all secondary parts, and parts in which stiffness rather than strength was desirable, are of carbon steel. The method of differentiating between the various members of a structure in accordance with their function has become general practice on important structures where such a procedure is warranted.

Physical Limitations of Higher Strength Steels

In considering various grades of steel for structural purposes the physical limitations of higher strength steels should be given attention because they will be reflected in the cost of the fabricated and erected steel.

Fabricating Problems.—All shaping and forming of metals is performed either by heating the material to a plastic state, or by exerting force on the cold metal. Within certain limits, structural steel is usually shaped by suitable devices at shop temperature. Cold-shaping is more convenient,

speedier, and more economical. Practically all straightening of plates and bars is done in the cold state. Cold-shaping and straightening of steel are based on the principle of applying force through a motion of the forming device so that the elastic limit is exceeded and the metal enters the plastic state and, therefore, follows the shaping tools into the desired form. Without exceeding the elastic limit, the metal, of course, would spring back into its original form. The fabricator is well aware of the fact that the higher the elastic limit of the material, the more force will have to be exerted to pass it and the stronger the forming equipment will have to be. Shaping and straightening will be more difficult and tools will break more frequently. It will also require more time because the higher elastic limit steel will need to pass through the straightening rolls a greater number of times.

It should also be remembered that the stronger steels require a more careful procedure in the pouring and generally more ingots are discarded. Their rolling temperature has a more limited range, and some, if not all, of the higher steels require protection from cold blasts and drafts during the rolling process. The finer the material the more care it requires; the stronger the steel, the more difficult it is to form.

Distribution of Stress Among the Rivets in a Joint.—Another matter that deserves serious consideration is that of stress distribution in riveted joints. When a relatively wide plate is spliced to another plate by the common riveted connection, the question arises as to whether the stress is distributed uniformly through the rivets from one plate to the other. The phenomenon of splicing steel by a group of rivets is by no means simple. Like most phenomena encountered in practice it has been made simple by an assumption which, although not correct, makes it possible to approximate the facts roughly and to proceed with the task of splicing the members. The assumption is that all parts, the plates and the rivets, are rigid members and that a stress, uniformly or continuously distributed in a plate, will transmit its total force to the connecting rivets and these, when symmetrically arranged, will transmit the force to the second plate in the same symmetrical and uniform distribution. This assumption dates from the time when stresses were low and the elastic properties of metal were ignored. Engineers know that such uniform distribution does not prevail in the plates near the rivets; the originally distributed stresses meet their resistance in the rivets and rush to them to overcome it. Concentrations of stress are formed around the rivets which sometimes exceed the elastic limit. At that time a system of equilibrium establishes itself between the plates and the rivets by the mutual yielding of each; where the intensity of stress is too great for the elastic limit, it passes (for the time being) into the plastic state. The phenomenon of stress distribution through rivets still requires much study. Engineers are aware of the problem and, at present, are devoting their attention to it. One aspect of the problem, however, is quite clear: The higher the elastic limit of the material the greater may become the concentration of stresses and the less uniform will be the distribution of stress in the plates. In fact, too high an elastic limit may lead to such a high concentration of stress as to cause incipient

failure. With the present knowledge of the phenomenon the prognosis may be made that the efficiency of riveted splices is not in direct proportion to the increase in the yield point of the material. The higher yield point, therefore, cannot be fully utilized in proportioning the connection.

What has been stated herein, concerning stress concentration on rivets, of course, holds generally true for all "hard points"; that is, for all places where the material encounters concentrated resistance. The same phenomenon is developed where the shape and area of the material change abruptly and the flow of stress is contracted and "choked." In such cases, the same as in riveted joints, it is desirable that the initial distributed stress be low and that the material should be able to adjust itself to the conditions. The development of photo-elastic studies in recent years has demonstrated, forcefully, the phenomenon of stress concentration.

The writer has dwelt at some length on the physical limitations of higher strength steels with the intent to emphasize the restrictions that may have to be imposed and to dampen the enthusiasm that the stronger material is likely to evoke.

Uses for High-Strength Steels

Modern economic, social, and technical development has brought people closer together in space and time than ever before in human history. The co-ordination of human efforts has brought together Man and mechanism. In the incessant endeavor to satisfy the ever-increasing demand for physical goods and to make the comforts of life accessible to the many, structures of great magnitude and capacity often become desirable as connecting links in production and distribution. Such structures and machines sustain heavier masses, move more freight, cross large rivers and wide bays, and lift up their spires into the skies. Because of their functions and of their magnitude they must sustain great forces, and materials of high strength are required for their construction.

Bridges with long spans require large members and, therefore, weigh more. Their stresses can be sustained by high-strength steel, with less material and at a lower cost. Tall buildings for special purposes, such as the Palace of Soviets, in Moscow, U. S. S. R., are subjected to high wind pressures and stronger steel will find its proper application for the skeletons of such structures. Derricks of large capacity and long-span cranes, and bridge members which are utilized during the erection of the structure and sustain great forces, may profitably be built of steel stronger than the common carbon steel. Wherever the demand for sustaining great forces will appear, necessity will compel the use of stronger steels independent of its economy. Where technical conditions of fabricating shops and facility of erection in the field will demand lighter sections and lighter members, higher strength steels will be applied. Their widest field of application, however, is in the reduced cost of the structure.

Engineering Economy.—In discussing the economy of large structures several interpretations and meanings of the word should be considered.

There is an economy inherent in the design of a structure which is part of the play of forces acting in the structure under load and of the arrangement of the sustaining members and their suitability to perform the functions which the behavior of the structure will demand. This economy is a measure of the mechanical fitness of the structure for its intended purpose. It is independent of the market price of the material and labor required to build it, and may well be designated as structural economy. Structural economy forms only a part of the engineering economy of a structure. The latter consists in utilizing for its various members such materials and shapes as will result in the lowest cost of the completed structure.

There is also another very important aspect of economy, namely, financial economy. In this economy enter considerations of the amount of capital invested in the project and the prospective life of the structure. It depends much on the prevailing rates of interest. It also is based on the economic organization of society in a given country. It is largely, however, through the development of engineering economy that higher strength steels will find their application and the field of stronger steels will be extended. The expansion will depend largely on the cost to the mills of the ingredient alloys which enter into the higher strength steels. It will depend on whether these alloys will be plentiful in the market or will be controlled by monopolies. It will also depend on the available processes of manufacturing and fabricating the stronger metals.

Improper Use of High-Strength Steels.—To realize engineering economy in structures fully, and to produce them at a lower cost, careful consideration must be given to the desired result in each case. Not only the first cost, but also the behavior of the structure under load, and its maintenance, should be weighed carefully.

A number of large bridges have been cited for which higher strength steels have been used, and an attempt has been made to give the reasons that have guided the engineers to adopt these steels. A few telling examples, out of many, will be brought to show instances in which higher strength steel has led to improper or uneconomical design.

In one instance an arch bridge of 140-ft span, of the plate-girder type, was designed. Its total steel weight was less than 200 tons, about one-half of which was of silicon steel. The arch ribs were made of 36 by $\frac{3}{8}$-in. webs and four flange angles, 6 by 6 by $\frac{1}{2}$ in. The ratio of the unsupported depth of plate to its thickness was 65. The web-plate was in compression and was manifestly too thin against buckling; it had to be thickened. Instead of using silicon steel, standard carbon steel should have been used. In general, it is not economical to use silicon steel for a short span of 140 ft. This case illustrates the physical and economical limitations in the use of silicon steel.

In another instance the designers evidently were aware of the buckling limitations of carbon steel and desired a compact section. They designed it, however, in an impractical manner. A highway bridge of the cantilever type was planned with a main span of about 800 ft. The chords were of

silicon steel and of the enclosed box type with angles turned in. The outside dimensions were $18\frac{1}{8}$ by $24\frac{1}{2}$ in. The minimum section carrying stress was composed of one top cover, 18 by $\frac{5}{16}$ in., four angles, 4 by 4 by $\frac{5}{16}$ in., two web-plates, 24 by $\frac{7}{16}$ in., and one bottom cover, 18 by $\frac{3}{8}$ in. In the maximum sections the top cover-plates and the upper angles were thickened to $\frac{5}{8}$ in., the web-plates to $\frac{7}{8}$ in., two side-plates, 15 by $\frac{3}{8}$ in., were added, and the bottom angles were made 4 by 4 by $\frac{3}{4}$ in. For riveting purposes and for painting, manholes, 8 by 24 in., were provided in the bottom cover-plate. The clear inside dimensions were thus about 12 by 20 in. for the heavy section. It is very difficult, if not dangerous, at the large joints, to put a man in these chords to do field riveting. The small box-sections also prove to be expensive both in the shop and in the field. Notwithstanding the constricted section of the chord, the ratios of depth to thickness of the thinner web-plates are somewhat high for silicon steel. In this case larger dimensions of chords would have provided more accessibility for riveting and painting, and the use of carbon steel would have furnished well-designed and economic chord sections at a lower total cost.

A third instance of the uneconomical use of stronger steel consists in using silicon steel indiscriminately at random points in carbon-steel trusses.

In a fairly large bridge of the cantilever type there is a suspended span of 150 ft. All members in this span were designed of carbon steel except one bottom-chord section which was made of silicon steel. It was made of two web-plates, 20 by $\frac{1}{2}$ in., and four angles, 4 by 4 by $\frac{7}{16}$ in. By thickening the material to about $\frac{5}{8}$ in., this section could also have been made of carbon steel and the total cost of the section would have been less. In this case a critical review of the material used would have disclosed to the engineer at once that the introduction of silicon steel in one chord section was not good economy.

It may be well to point out that, although for a large tonnage fabricated silicon steel in place will command a differential of \$12 per ton, for smaller tonnage the extra charge will be greater. Where the weight of each individual shape is less than 20 tons, the extra charge is \$15 per ton. For plates smaller than 36 in., the differential is \$10 and for plates larger than 36 in., it is \$15. Silicon-steel bars of various sizes in quantities of less than 1 000 lb will cost \$25 to \$35 per ton extra. The higher extra charges for the smaller quantities are due to the care required to segregate and follow the small quantity of the special steel and to the difficulty of replacing from stock any items rejected during fabrication.

The foregoing examples of actual practice have been cited not in the spirit of criticism, but with the purpose of calling the attention of engineers to the possibility of errors of judgment and oversight and to establish warning signals.

Conclusion

The progress of the more civilized part of humanity has led to the growth and concentration of population in large cities. At the same time,

it has created an enormous demand for rapid and economical transportation of large masses of freight and of great numbers of passengers. Among other efforts it has created a demand for bridges of large capacity over wide rivers. Engineers in their untiring search to meet the demands of growing communities and industry have successfully made use of a few higher strength steels which they have developed and found available. The field for the application of such steels appears to be extending and standardized specifications have been established for the use of manufacturers and engineers. The varieties of alloy steels of higher strength are by no means limited to the few which have actually been used in structures in the United States. At present, a number of high-strength steels have been developed and are available on the market. Many more will be produced to meet the demand. It is the task of the engineer to follow the development of materials and to utilize them to the best advantage. At the same time, it is incumbent on him to study the physical and economical limitations of the higher strength steels and to determine carefully in each case the best advantage that may be derived from their use.

Much effort is being made at present to increase the pay load on freight-carrying vehicles. While materials are being developed to reduce the dead weight of cars and trucks, the pay load, that is, the weight of the freight carried, will be increased and the tendency is strong to increase the total weight of the loaded vehicle. To be able to compete with freight transportation on highways, railroad officials aim to reduce the cost per ton carried. With the present tendency to higher cost of labor, the cost per ton carried must be held down by mechanical efficiency. The result will be heavier trains pulled by heavy locomotives, for which heavier bridges will be required. This is evidenced by the latest specifications for railway bridges. On the other hand, the growth of automobile traffic has made long-span bridges economically feasible. Engineering science, the art of metallurgy and fabricating technique, and erection skill will make it possible to build them economically in greater numbers. During the two decades, 1916 to 1936, the span of bridges has been more than doubled. Engineers have attained a span of 4 200 ft and will surpass it in the future. The use of stronger steels for structures has been intimately connected with the progress of bridge construction and will so continue.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
PAPERS

STRUCTURAL APPLICATIONS OF STEEL AND LIGHT-WEIGHT ALLOYS

A SYMPOSIUM

PAGE

IV.—LIGHT-WEIGHT STRUCTURAL DESIGNS:

The Application of Stainless Steel in Light-Weight Construction.
By E. J. W. RAGSDALE, ESQ. 1304

Structural Application of Aluminum Alloys.
By E. C. HARTMANN, ASSOC. M. AM. SOC. C. E. 1313

Magnesium Alloys and Their Structural Application.
By A. W. WINSTON, ESQ. 1329

THE APPLICATION OF STAINLESS STEEL IN LIGHT-WEIGHT CONSTRUCTION

By E. J. W. Ragsdale[87], Esq.

Synopsis

In a broad sense, this paper covers the applicability to light construction of the two major types of stainless steel and then describes briefly the general problems of design, fabrication, and usages. Questions of plate stability, efficiency of spot-welded connections, and the economic phases of a relatively costly material are lightly touched upon. Although these seem to merit a more expansive treatment, the danger of such expansion is always that the paper may be thrown out of balance.

Stainless Steels

The structural engineer need concern himself little about the metallurgy of the materials he uses. His purpose is to define requirement, and then to select the materials best suited to it. That selection is made on the basis of established properties, which become accepted values. The only concern is that these properties be not disturbed by continued use or by the processes of manufacture. So much appreciation of metallurgy must the structural engineer have.

When consideration, however, involves the innumerable alloys listed under the name of stainless steel, a mere appreciation of metallurgy no longer suffices. The structural engineer had then best invite the close co-operation of the metallurgist. No less than a dozen different alloys present themselves. Aside from a pronounced resistance to corrosion, they have little in common. Some owe their strength to heat treatment, others lose it thereby. Some are inherently stainless, whereas others are only stainless under certain conditions.

Fortunately, a structural application of stainless steel has already developed, and, through this, the choice of alloys becomes narrowed. Outstanding is a chrome-nickel alloy (18–8)[88], that has been described in detail by Mr. M. J. R. Morris, in this Symposium[89]. As a poor second is a straight chromium steel with from 12 to 17% chromium content. Requirement and the process of fabrication will determine the applicability of each. The first owes its high physical properties to cold working, and, by the same token, may lose them by welding. The other improves by heat treatment, which treatment automatically corrects the effects of any previous welding operation. Therefore, it becomes quickly apparent that the feasibility of heat treating a finished structure may determine the choice. Further considerations will be cost and strength. The cost of the chrome-nickel steel is about 50% greater than that of the chromium

[87] With Edward G. Budd Mfg. Co., Philadelphia, Pa.

[88] Symbols in parentheses denote the trade designation by which the alloy is commonly recognized.

[89] See Table 4, Items Nos. 5 to 10, inclusive.

steel, and, at the same time, it is cold rolled to a 50% greater strength. An even greater advantage, however, lies in the susceptibility of chrome-nickel steel to good and cheap fabrication.

Perhaps a very general distinction may be made by stating that the chrome-nickel steel is better suited to the requirement of light-weight construction, whereas the chrome iron offers a corrosion-resistant substitute for carbon steels in more conventional design.

Design Problems

Light-weight construction has as its premise a more effective use of a more effective material. This indicates a wider spread of material, and, at the same time, a lesser material requirement. Both result in a reduction of the usual thicknesses, and herein lies a problem.

It is an old and convincing trick to illustrate structural stability by trying to stand a piece of paper on edge and then showing how this same sheet increases in load-carrying capacity as it is rolled into cylinders of smaller and smaller diameter. The limit is finally reached when the tube becomes so small as to lack column stiffness. A compromise position then results in a larger tube and a stiffened wall. This thin wall may be strengthened by structural members, by longitudinal corrugation, or by both. The ultimate effectiveness is reached when the column becomes a series of vertical members, mutually cross-braced. The wall has disappeared.

This question of wall or plate stability, however, has already been investigated, especially in connection with bridge construction. It becomes only more acute as the ultimate strength of metals is raised from 75 kips per sq in. to twice that value, and when thicknesses are measured in thousandths of an inch rather than in quarters of an inch. In this case, the question of relative flatness complicates an already complex mathematical treatment. The obvious solution becomes empirical and toward this end a curve on the so-called "flat pitch ratios", has been developed with which it is possible to compute the stability limit in compression of any closed section that is made up of a series of flat surfaces. The "flat pitch ratio", q, is the relation between the unsupported width of a flat surface and the total wall thickness; that is,

$$q = \frac{b'}{d} \qquad \text{.............................(5)}$$

Taking this value for the widest face of the section, Fig. 36 will show at what percentage of the ultimate strength of the metal, that particular face begins to undulate under compressive load. (The abscissas are plotted as ratios of ultimate compression stress, T_c, to ultimate tensile stress, T_t.)

Quite obviously, therefore, a metal of high tensile strength must be formed into sections that permit a stressing commensurate with its superior strength. An I-beam, for instance, may develop stresses up to the limit of mild steel, but if made of a steel having a working stress three times greater, the beam might collapse through instability long before that stress was realized. Therefore, the sections suitable for one metal cannot be extended arbitrarily for the use of another. Light-weight construction therefore, has brought into

being a series of new and more efficient sections. These sections are largely of the closed-box type, or efficient open sections which are locally stabilized by stiffeners. Some are formed by rolling from a strip, whereas others are fabricated.

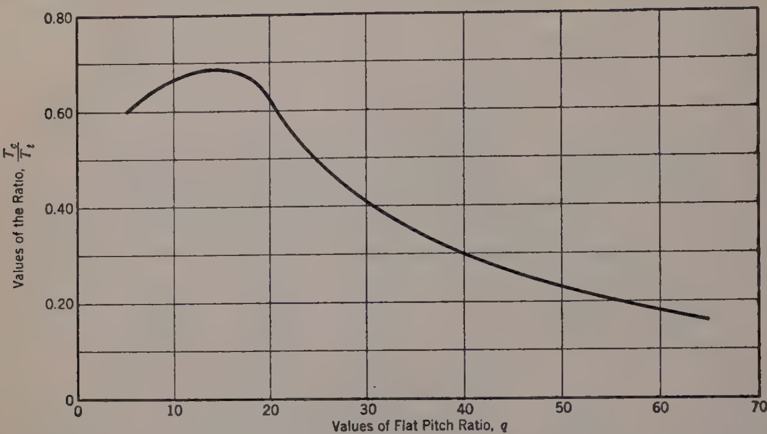


FIG. 36.

Fig. 37(*a*) and Fig. 37(*b*) are compression truss members, and Fig. 37(*c*) and Fig. 37(*d*) represent floor-beams. All of them (and, in fact, most sections in use) will develop at least 100 kips per sq in. on the compression side;

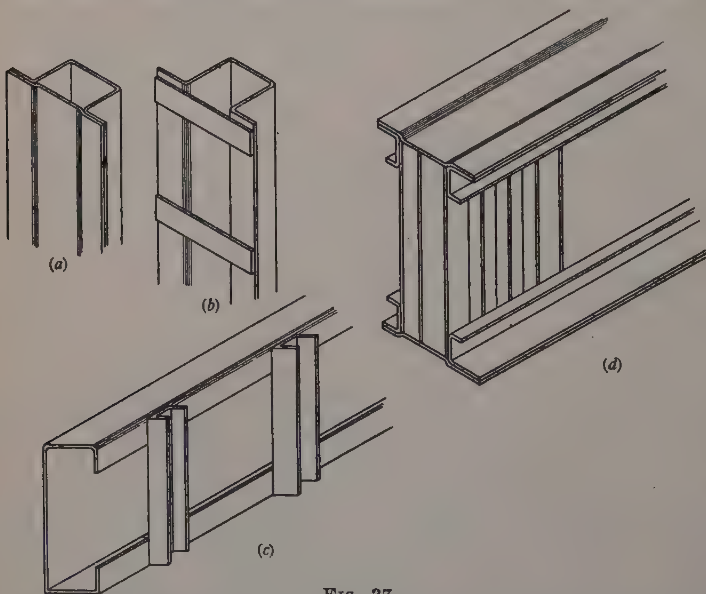


FIG. 37.

that is, when they are made of chrome-nickel steel, having a tensile strength of 150 kips per sq in. The prevalence of square-faced sections is due primarily to the relative ease of subsequent assembly.

Computed on a weight-to-strength ratio, such sections in material of high tensile strength, will be found to be from seven to ten times better than the conventional profiles of mild steel. One-half the advantage comes from the superior metal strength, and the other half from a more effective placement of that metal. Of course, this ratio relates to relatively small sections, such as 6-in. channels or I-beams, and becomes less apparent when compared to the efficient girders, such as obtain in bridge construction.

Efficient and strong as these individual sections may be, however, they are useful only to the extent that their connection, one with another, is equally efficient. Attachment becomes a problem, both as to area and avoidance of eccentricities. In fact, the same care and design exercised in large bridge structures must be interpreted into use for smaller and lighter ones of high-strength metal. Double gusset--plates, nested member ends, and efficient bonding, or joining, become necessary. These requirements again demand close attention to sequence of assembly and to accessibility of welding or riveting. The latter is complicated to the extent that the structure may be small or truly efficient. It is further aggravated often by lack of appreciation of shop procedure. A seemingly simple connection on paper, may baffle a Houdini in assembly.

Even when a type connection may be decided upon, there still remains, however, the question of providing sufficient bond. This bond may be by rivets, or by the fusion of faying surfaces as in welding. In either case, the function of the bond is always to resist straight shear. A definite practice has been established for this function in rivets and mild steel. It is not translatable immediately into high-strength material for the simple reason that rivets of correspondingly stronger physical properties cannot be applied. The requisite increase of shear is more aptly obtained by welding. This offers possibility of a 100% efficient joint. It involves, however, many new considerations of design and shop technique.

Welding

Nothing is simpler than to calculate the strength of a welded connection, and few calculations can be more misleading. The difficulty lies in the assumptions. In the first place, one assumes that the weld is good, and, in the second, that the primary forces still govern. An improved welding technique has done much toward establishing welding certainty, but the relief of heat-induced stress concentrations in and about the welded area remains a problem, the solution of which differs for each individual case.

Although both chrome-nickel steel (18–8) and the chromium steel can be gas welded, arc welding is becoming the more favored of the two processes. It must be recognized, however, that either procedure is a severe heat operation and that these alloys are peculiarly responsive to such treatment. No responsible arc welding should be done, therefore, without subsequent heat treatment. This is not with a view to relieving stresses so much as to correct an impaired metallurgical condition in the weld and the adjacent metal. Since the chrome irons are usually improved by heat treatment, not only is the weld corrected,

but the entire structure becomes benefited. Arc welding is strongly recommended for the straight chrome series, but the limitation of heat treatment also confines the practice to such structural members as may be susceptible to this treatment. These members are usually small and simple pieces, such as will not warp under the application of a high heat.

With chrome-nickel steel there are two difficulties attending arc welding. In cooling through the critical temperature gradient of from 1 200° to 1 600° F, carbides are precipitated. These carbides can only be returned to solution by heating to more than 2 000° and quenching. Then, again, chrome-nickel (18–8) owes its superior physical properties to cold working, and these properties are automatically reduced by either the heat of arc welding or the corrective re-heat treatment. Since this metal in the annealed state only shows an elastic limit of about 34 kips per sq in., its utility in highly stressed members is, therefore, defeated by arc-welded assemblies.

Thus far, the happiest solution for the joining of chrome-nickel steel appears to be found in a special system similar to that of ordinary spot welding. In principle, however, it differs by the establishment of such controls of current, time, and electrode pressure that the metal remains metallurgically unimpaired. This is made possible by the discovery that both carbide precipitation and the annealing of the metal adjacent to the zone of fusion are functions of the time of heat application. With a quick application of heat and a rapid cooling, neither of these undesirable phenomena appear. Welds may be made in as brief a time as one-half cycle, or $\frac{1}{120}$ sec. Rarely does the time of current dwell exceed ten cycles.

However, the process does not correct an inherently insusceptible condition, such as obtains in the chrome irons. In the latter case, regardless of the method of fusion, the bond is coarse-grained and brittle. It can only be corrected by subsequent heat treatment.

Chrome-nickel steel, on the other hand, is strangely susceptible to good welding. Its every property seemingly conspires toward this method of bonding. Recalling that in resistance welding the heat generated is expressed as $I^2R\ t$, the item, R, which is the electrical resistance imposed between the electrodes, assumes basic significance. With corrodible metals, R sums up the resistance of the body of the metal as well as that of the oxided surfaces. The latter not only varies widely with the extent of oxidation, but is usually many times greater than the resistance of the pure metal itself. Therefore, R is not a constant, and no matter how accurately controlled current and time may be, the resultant heat will vary. With stainless steel, the surface resistance is small and constant. Furthermore, the basic resistance is about six to eight times greater than that of mild steel. Accordingly, the current or time required for a given fusion is less.

Fusion initiates at the inner contacting surfaces of two or more sheets, and progresses outward in proportion to the total heat applied. Such is the heat-conducting property of the electrodes, however, that it can reach the outside surfaces only in the case of a badly burned weld. The shape and depth of the fused button are important. Although the diameter determines the area

in shear, the penetration of the fusion into each adjoining sheet holds these sheets together much the same as the heads of a rivet. A shallow weld lacks tenacity; one that extends entirely through the sheets will invite corrosion. The control of weld shapes lies in the proper setting of the welding machine, which, in turn, is determined as a result of many tests with sectionalized and etched specimen welds.

Having established a proper welding practice, the next step is to insure a strict observance of such practice, because when welding becomes an element in structural procedure, the same standard of reliability is required of it as of the materials to be so bonded together. This seemingly ever-present problem in welding of any kind, has been solved in the special system herein cited, by use of a simple, but unique device for integrating the function, $I^2R\ t$, for each weld made. Any departure from the allowed tolerance rings a bell in the welder, and the result of each weld made or failed is recorded on a tape. Welds cannot be judged from the outside and, without the device mentioned, the operator is really working in the dark and beyond the scope of inspection.

On the other hand, how to make a weld is one thing; what it is worth is another; and therein is found again a remarkable property of chrome-nickel steel. The fused metal has been subjected to the very treatment recommended for annealing to a dead soft condition. Its tensile strength is 90 kips per sq in. More remarkable, however, is the fact that its shear strength is also 90 kips per sq in. This means that one weld has twice the strength of a rivet of like diameter, and since welds can also be placed closer together than rivets, a much more efficient joining results. Single lap-joints can easily show 85% efficiency, against 60% in good riveted connections. Just what stress distribution takes place in and about a welded joint, however, remains a neat problem to engage the ingenuity of some physicist.

When the conventional rivet hole is replaced by a metal softer than the surrounding sheet, and when this soft metal also assumes resisting capacity in tension with the sheet, it is difficult to visualize the proportion and paths of the lines of stress. However, the philosophy of welding is so engaging that one is likely to overlook the only practical consideration which makes this form of welding at all applicable to high-strength chrome-nickel steel (18–8). The point seems never before to have been emphasized; that is, that welding pressures can be used which are sufficient to draw the sheets closely together. In mild steel no such pressure is required because the steel is soft. Were such pressure necessary, the electrodes would break through the surface scale and, in thus reducing the resistance, would make for no weld. This phenomenon is known. With chrome-nickel steel, the electrode pressure is often more than 2 kips for a $\frac{1}{4}$-in. weld in two 16-gage sheets.

The overlapping of individual spots to form a gas-tight joint is known as seam welding. The same principle applies in this case, but pointed electrodes are usually replaced by rollers. So many considerations are involved, however, that this form of welding must be approached with a full appreciation of the difficulties. The problem is too specialized to be more than mentioned in this paper.

Applications

Perhaps the first structural application of chrome-nickel steel was in connection with aircraft. The fact that it could be cold rolled to tensile strengths greater than 200 kips per sq in. and used in thin sheets without fear of corrosion, had a strong appeal. Some research has also been made in connection with bridges. This has been confined mostly to roadways and decorative effects. A much wider application awaits development. Weight reduction also has its place in this connection, not to mention the elimination of the high cost of painting in difficult and inaccessible places.

The development in marine structures has been far wider. It is generally known that warships have become weight conscious almost to an extreme limit and that deck-houses, masts, smoke-stacks, and bulkhead doors constitute only a beginning in the program. Several million pounds of chrome-nickel steel have already been used in this connection and the ultimate is not yet in sight. Nor is the interest confined to naval vessels; the merchant marine has also discovered that stainless steels have their definite uses. The installation of storm windows on the S. S. *Normandie*, for example, is but a modest beginning of the use to which the material can be applied economically and well. Add to the high strength and to the non-corrosive property of chrome-nickel steel a heat resistance almost as remarkable, and it becomes apparent that this type of stainless steel fits into the marine requirement as has no material of the past. Although the cost causes an initial hesitancy, this will be overcome as soon as ideas become adjusted to true economic worth. The present (1936) cost of chrome-nickel steel[90] is about 30 cents per lb. No wonder that structural engineers may hesitate, especially when alloyed steel for bridges is only a fraction more than 1 cent. Translate weight into motion, however, and metal costs become ridiculous. Dirigibles and some heavier-than-air craft cost as much as $20 per lb. Designers of automobile trucks and trailers consider that a pound saved and converted into pay load may be worth $1 per yr. Every extra pound in a Diesel-electric train carries 30 cents for additional power and equipment cost.

Most spectacular have been the developments in the application of chrome-nickel steel to railway cars, and, by the same token, the most publicized. Rarely have the results of research been more timely. With the railroads struggling under the weight of their own rolling stock rather than that of pay loads, and with a depression to emphasize the condition, an improvement was indicated—not just a slight betterment, but a radical one. It was typified by the first *Zephyr* train, the phenomenal performance of which has stimulated efforts not only along this line of construction, but in other directions as well. The field seems wide open for any material superior to mild steel. Ingenuity of design vies with ingenuity of cost argument, and the main objective is often lost from sight. This object in the case of the *Zephyr*, was to reduce the weight of a railway car to substantially one-third that of a heavy passenger coach, and to do so without sacrifice of strength or comfort. The

---

[90] See "Stainless High-Alloy Structural Steels", by M. J. R. Morris, Table 4, Items Nos. 5 to 10, inclusive, pp. 1202–1203.

three cars of the *Zephyr*, without the power plant, weighed 160 000 lb. Conventional railroad coaches weigh from 130 000 to 160 000 lb. So nearly was that part of the objective attained. Then, with a reduced mass, so were destructive forces also reduced. In a million miles of high-speed operation by these *Zephyr* type trains, and not without exposure to damage, there has been neither structural failure nor weakness. A more severe test of a comparatively new structural metal and a new method of fabrication can scarcely be imagined. It assumes a significance which should be more convincing than the listing of physical properties, or the discussion of welding merits.

Just what share of future railroad building will be enjoyed by stainless steel cannot, of course, be predicted. More certain seems the prediction of a prominent railroad consultant, to the effect that the last heavy railway car has been built. With this great industry becoming weight conscious, and with an inevitable building program ahead, the only limit to the part that can be played by stainless steel appears now to be one of facilities and organizations capable of handling it.

Stainless steel will open other lines of possible utility, as the knowledge of its uses is extended. An excellent book[91] published in 1935, contains a description not only of the properties and methods, but of the width of the fields already attempted. That this field is so sparsely covered reflects merely upon a lack of applied research; for instance, the structural application of chrome-nickel steel owes its development almost entirely to the efforts and enthusiasm of one man. The staff of engineers which he could afford to divert from his normal business in order to prosecute this development was pitifully small. The temptation to follow each new promising lead has been great, for they are promising—all of them. They require development, and this needs only time and enthusiasm, for the foundation work in stainless steels has now been done.

Acknowledgment

The writer is indebted to his employer, The Edward G. Budd Manufacturing Company, for the use of the basic data required for this paper and for permission to present Figs. 36 and 37.

[91] "The Book of Stainless Steels", Edited by Ernest E. Thum, Am. Soc. of Metals, pub., 1935.

STRUCTURAL APPLICATION OF ALUMINUM ALLOYS

By E. C. Hartmann,[92] Assoc. M. Am. Soc. C. E.

Synopsis

The aluminum alloy most frequently used for structural purposes is described in this paper from the standpoint of the designing engineer. The composition, properties, and forms available are given. Shop fabrication and workmanship are described to show that the practice differs very little from that for ordinary steel fabrication. The recommended design stresses for tension, compression, shear, and bearing are included, and the problem of designing to take advantage of the lighter weight is discussed. The paper is concluded with a brief study of the economics of the use of structural aluminum, indicating the extent of the extra costs and in what fields such extra costs may be justified.

Characteristics of Structural Aluminum

The engineer interested in minimizing dead weight will find commercially available to-day more than a dozen aluminum alloys which have been used structurally. Each of these alloys, some wrought and some cast, is fitted for some particular field of structural work by virtue of its properties. The selection of the proper alloy to be used in any particular case, involves a number of problems outside the scope of this paper, and, therefore, in order to avoid confusion, the writer will confine himself principally to the wrought aluminum alloy which has been most commonly used in the structural field. This alloy (17S-T)[93] is of the duralumin type.[94]

It contains[95] nominally 95% of aluminum, 4% of copper, and 0.5% each of manganese and magnesium. It is a heat-treatable alloy and is never used structurally except in the heat-treated condition, when it develops the following typical properties:

Weight, in pounds per cubic foot................	174
Ultimate tensile strength, in kips per square inch..	58
Yield strength, in kips per square inch...........	35
Percentage elongation in 2 in..................	20
Ultimate shearing strength, in kips per square inch.	35
Shearing yield strength, in kips per square inch....	20

[92] Research Engr., Aluminum Co. of America, New Kensington, Pa.

[93] The symbols in parentheses denote the trade designation by which the alloy is commonly recognized.

[94] Tentative Specifications B78-33T and B89-33T, Am. Soc. for Testing Materials.

[95] "Light-Weight Structural Alloys", by Messrs. Zay Jeffries, C. F. Nagel, Jr., and R. T. Wood, Table 6, Item No. 6, p. 1217, and Table 8, Item No. 6, p. 1225.

Modulus of elasticity in tension and compression, in pounds per square inch........................	10 300 000
Modulus of elasticity in shear, in pounds per square inch	3 800 000
Coefficient of expansion, in degrees per degree Fahrenheit	0.000012
Poisson's ratio	0.33

The yield strength is defined as the stress which produces a permanent set of 0.2% of the initial gage length[96] (see Fig. 38).

The guaranteed minimum properties for this material (in kips per square inch) are, as follows:

Description.	Ultimate tensile strength	Yield point strength.
Plates.................	55	32
Structural shapes.......	50	30

Fig. 38 is a typical tensile stress-strain curve for the material.

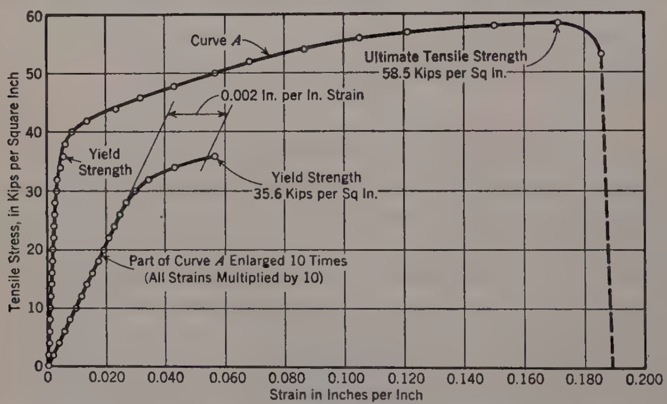


FIG. 38.—TYPICAL STRESS-STRAIN CURVE FOR STRUCTURAL ALUMINUM (DATA TAKEN FROM 0.5-INCH ROUND SPECIMEN.)

Structural aluminum is produced in the form of sheets, plates, structural shapes, rods, bars, rivets, tubing, and forgings. In the form of plates, widths as great as 120 in., thicknesses as much as 1 in., and lengths as great as 35 ft are available, with a weight limitation of 2 kips in any individual piece. In shapes, structural aluminum (17S-T)[83] is available in the form of angles as large as 6 by 6 in., channels as deep as 12 in., I-beams and H-beams as deep as 8 in., and Z-bars as deep as 5 in. Many of the structural shapes are available in lengths as great as 85 ft, but some of them have a length limitation of 35 ft.

Fabrication of Structural Aluminum

Structural aluminum can be fabricated in the ordinary shop with practically no departure from commonly accepted good practice for other

[96] Specification E8-33, Am. Soc. for Testing Materials.

structural metals. The material can be cut by shearing or sawing, the use of the burning torch being prohibited, as it would be for any other heat-treated material. It can be sheared with equipment ordinarily used for steel. Both band and circular saws are used for sawing, and a lubricant or soluble cutting compound is recommended. High-speed metal-cutting band-saws, or heavy-duty wood band-saws with blade speeds of about 5 000 ft per min, are quite satisfactory. Circular saws with hollow ground blades are satisfactory for light work, but for heavy work the teeth should be swaged in order to provide clearance and prevent over-heating. Coarse-toothed blades should be used for both band and circular saws, and the teeth should have no top rake. With circular saws, a peripheral speed of 12 000 ft per min is recommended for the best results.

Structural aluminum can be worked cold to a limited extent. To obtain the best results in cold bending, it is necessary to have all tools in good condition, and to minimize friction between the tools and the metal that is being formed. The metal itself should be free from nicks and scratches, particularly along the edges, and the radii for all bends should be as large as practical.

For difficult forming the metal should be heated, but this should not be undertaken unless equipment for accurate temperature control is available. Heating the metal to a temperature of 300° F improves the cold-forming characteristics somewhat and does not seriously affect the properties of the finished piece, provided the metal is heated less than 30 min. For more difficult forming, however, it is usually necessary to heat the piece to the heat-treating temperatures, 930° to 950° F. Such heating, of course, reduces the mechanical properties of the metal unless it is followed immediately by a suitable quench. In the ordinary heat treatment of the metal this quench is provided by immediate immersion in cold water, but since such a quench usually results in objectionable distortion, it is not generally utilized in the fabrication shop unless some means of straightening the finished piece is available. The more common method of providing a quench in the fabricating shop is to transfer the metal quickly from the heating medium to the forming die, which should be cold and of sufficient size to chill the metal thoroughly and suddenly during the forming operation. Since the metal is thoroughly supported and held in alignment by the die, the finished piece is not distorted.

It is clear from the foregoing that, in order that structural aluminum may be hot-formed successfully, it is necessary to have suitable heating equipment with accurate temperature control. Some fabricators of structural aluminum have found it desirable to install such equipment, but in other cases, where only a few pieces require forming, arrangements have been made to have such forming done at the plant of the producer of the metal prior to shipment. The engineer who appreciates the difficulties of forming and bending will make an effort, of course, to minimize the number of pieces that require such treatment, or he will try to limit such pieces to low stressed portions of the structure so that they may be made from one of the lower strength alloys having better forming characteristics than the particular structural aluminum alloy (17S-T), described in this paper.

Rivet holes in structural aluminum may be drilled, punched, or subpunched and reamed as desired, although drilled or reamed holes are preferable to punched holes. For reaming, the taper bridge reamer with spiral flutes is recommended.

Most aluminum alloy structures, particularly those involving the larger sizes of shapes and plates, are riveted with hot-driven steel rivets. The heating effect of the individual steel rivets is dissipated very quickly by the excellent thermal conductivity of the metal, so that the strength of the aluminum parts adjacent to the rivets is not affected adversely. Some fabricators have adopted the precaution of driving steel rivets at random, particularly in locations where they are closely spaced, to minimize the heating effect.

Aluminum alloy rivets have been used successfully in a number of structures, particularly in sizes smaller than $\frac{5}{8}$ in. which can be cold driven successfully. When it is necessary to use them in the larger sizes, the driving is done hot, and, again, it is necessary to have suitable heating equipment available. The hot-driving of aluminum rivets is very similar to the die-quench hot-forming operation previously described; that is, the rivets are inserted in the hole and driven immediately after removal from the heating medium so that a satisfactory quench is effected by the contact between the rivet and the relatively cold tools and metal around the hole. Some fabricators have successfully adopted the aircraft practice of driving cold rivets of structural aluminum (17S-T) in their semi-soft condition, immediately after heating and quenching, natural aging bringing the strength of the finished rivets up to maximum value over a period of three or four days. In general, however, the additional weight saved by the use of aluminum rivets has not been considered sufficiently important to cause them to be generally adopted instead of steel rivets.

The welding of structural aluminum, like that of most heat-treated materials, has not developed to the stage where it may be utilized to replace rivets. The heat of welding tends to anneal the metal adjacent to the welded area, greatly reducing the strength. When welding is necessary in aluminum construction, one of the lower strength alloys particularly suited for welding is used instead of the standard alloy treated in this paper (17S-T). This applies principally to tanks and light framework rather than to the larger structures.

Aluminum structures should be painted thoroughly, particularly when they are to be used in locations in which the corrosive conditions are severe or uncertain. The procedure generally recommended is to use a priming coat containing a substantial quantity of zinc chromate in a synthetic resin vehicle, applied to all surfaces of the structural parts and allowed to dry before assembly. All parts should be clean and dry, of course, before the application of the priming coat. Following the riveting operation, all rivet heads and adjacent parts should be touched up with the same primer. It is common practice to use at least two protective coatings of aluminum paint over the primer, and care should be taken to work the paint in well around the rivet heads.

No discussion of the fabrication of light-weight structures can be complete without proper consideration of workmanship. When aluminum alloys are

utilized in a structure, it is generally for the specific purpose of reducing dead weight, and, therefore, one may logically assume that great care will be exercised in the design to produce a given result with the least possible material. Any such design deserves and should receive the most careful attention from the standpoint of workmanship. This means, of course, that: (1) Tools should be kept in first-class condition; (2) that holes should be round and no larger than necessary; (3) that edge distances should be carefully maintained and all edges should be well dressed to avoid notches and roughness; and (4) that all material should be handled in the shop work so that surfaces and edges are protected from dents and other injurious accidental defects. It is important that all re-entrant corners be provided with suitable radii to avoid stress concentration, and that rivets shall not be over-driven. In short, the fabricator should observe all the well-known rules of good structural metal practice as set forth in standard specifications in order that the excellence of the shop work may be commensurate with the care exercised in the design, and may contribute to the weight saving expected by the designer.

Designing with Structural Aluminum

In preparing the design of any structure the engineer, of course, must be familiar with the properties of the materials being used and the behavior of those materials under different stress conditions. Unfortunately, it is not always possible for the designing engineer to obtain this knowledge through first-hand experience in actual handling and testing of the materials, and, therefore, he must rely on other sources of information. Generally, he depends for his knowledge of the behavior of any material on some prepared set of specifications which limit the working stresses in the structure to values which are known to be safe on the basis of available test results, theoretical studies, and the experience of those engineers most familiar with the material.

In the case of aluminum alloys, particularly the standard product (17S-T) herein described, a wide variety of tests is available indicative of the structural behavior of the metal, as well as a considerable background of experience in its structural application. Although it is beyond the scope of this paper to go into detail regarding either of these interesting phases of the development of structural aluminum, the writer and his associates have drawn heavily on both in the preparation of the following design stresses. Theoretical studies, such as those by Professor S. Timoshenko[97], have also played an important part in the selection of suitable formulas.

These design stresses represent a factor of safety of at least two against permanent set and buckling, and a factor of safety of at least three against ultimate failure. In each case an attempt has been made to express the allowable stress in such a way that the designer has maximum freedom from arbitrary limitations so that he may exercise his ingenuity to the fullest extent in obtaining maximum weight savings within the limits of safety.

For the basic allowable unit stress on structural aluminum (17S-T) members in direct tension, or on the tension flanges of beams and girders (net section), 15 kips per sq. in. is recommended. This value is one-half the

[97] "Strength of Materials", by S. Timoshenko.

guaranteed minimum yield strength for rolled structural shapes, and is less than one-third the guaranteed minimum tensile strength.

For compression on columns and other compression members having an effective slenderness ratio, $\frac{aL}{k}$, equal to or less than 81, an allowable stress (in pounds per square inch) is recommended in accordance with the following straight-line formula:

$$s_a = 15\,000 - 123\,\frac{aL}{k} \quad \text{(6)}$$

In Equation (6), L is defined as the unsupported length of the member, in inches; k is the corresponding least radius of gyration, in inches; and a is a factor describing the end conditions of the member. For both ends fixed, a is equal to 0.5, and for both ends pinned, a is equal to 1.0. Since few if any structural members have completely fixed ends, the value of a should rarely be less than 0.6. In framed construction most compression members have partly fixed ends, and an a-factor should be selected between 0.6 and 1.0, depending upon the end conditions.

For compression on columns and other compression members having an effective slenderness ratio, $\frac{aL}{k}$, greater than 81, the allowable working stress should be in accordance with the following formula:

$$s_a = \frac{33\,000\,000}{\left(\frac{aL}{k}\right)^2} \quad \text{(7)}$$

Equations (6) and (7) for compression are plotted in Fig. 39, and it will be seen that the straight-line portion is tangent to the curved portion, the

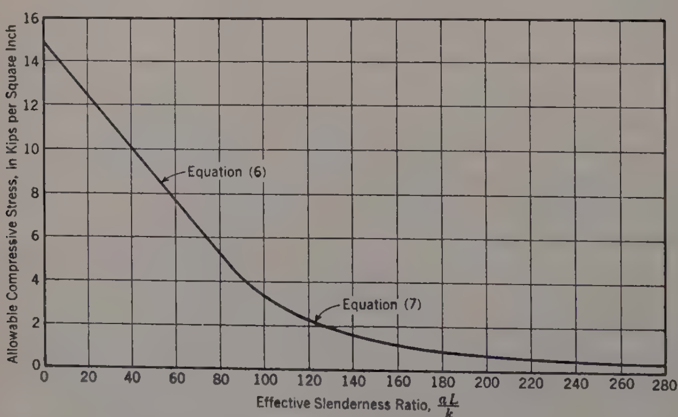


FIG. 39.—ALLOWABLE COMPRESSIVE STRESS IN ALUMINUM ALLOY (17S-T) COLUMNS.

latter being simply the well-known Euler formula with a factor of safety of three applied. This combination of formulas has been found to represent

fairly well the general trend of column tests on a large variety of structural aluminum compression members. It is used in the same manner as most other column formulas, inasmuch as it applies to the average stress on the gross cross-sectional area of the member at or near the center of the unsupported length. It will be noted that there is no rounding off of the column curve as it approaches $\frac{aL}{k} = 0$. In the writer's opinion it is much more logical to accomplish this effect by applying suitable restrictions to the allowable stresses on members which are likely to fail locally.

Flat plates or the legs, webs, and flanges of structural shapes tend to buckle locally when stressed in compression, particularly if such parts are fairly thin with respect to their unsupported widths. Because of the lower modulus of elasticity, aluminum alloys have less resistance to such buckling than steel, and, for this reason, it is more important, perhaps, that the allowable compressive stresses be defined more clearly than in the case of steel. In any event, it is far better to define the safe allowable stresses on such parts than arbitrarily to limit their dimensions because, as previously indicated, the designer should be allowed as much freedom as possible in working toward maximum weight savings.

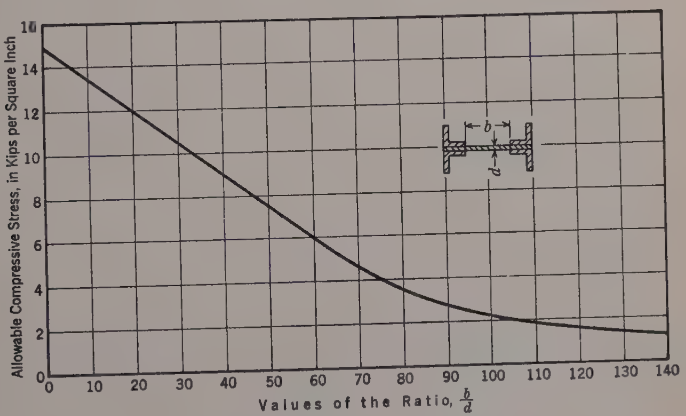


FIG. 40.—ALLOWABLE COMPRESSIVE STRESS ON ALUMINUM ALLOY PLATE SUPPORTED ON TWO EDGES

When flat plates buckle in compression, such buckling occurs in the form of local waves or wrinkles which are practically independent of the length of the member, provided such length is large with respect to the unsupported width of the plate. These local buckling failures in plates may be treated conveniently as local column failures of parts of the plate, using the column formula for the material, provided the proper "equivalent slenderness ratios" are used. From theoretical considerations and available test results the writer and his associates have evaluated these equivalent slenderness ratios for various cases as indicated subsequently.

For flat plates supported along two sides parallel to the direction of stress as in the case of the webs of H-shaped columns, or the compression cover-

plates of double-web box girders, the equivalent slenderness ratio of the plate should be determined, as follows:

$$\frac{aL}{k} = 1.2\frac{b}{d} \quad \text{.........................} \quad (8)$$

in which b is the unsupported width, and d, the thickness, both in inches. This equivalent slenderness ratio may be used in the column formulas, Equations (6) and (7), for determining the allowable compressive stress on such plates. The values thus determined are plotted in Fig. 40.

For flat projecting plates supported along one side parallel to the direction of stress, as in the case of the outstanding legs of single-angle, double-angle, **T**-shaped or star-shaped struts, the equivalent slenderness ratio of the projecting plate should be determined, as follows:

$$\frac{aL}{k} = 4.0\frac{b}{d} \quad \text{.........................} \quad (9)$$

in which b is again the unsupported width and d the thickness (see Fig. 41, Case 1). Tests have shown that single-angle, double-angle, **T**-shaped and star-shaped struts have very little restraint against twisting if any outstanding leg begins to buckle, and the coefficient, 4.0, in Equation (9) is selected on

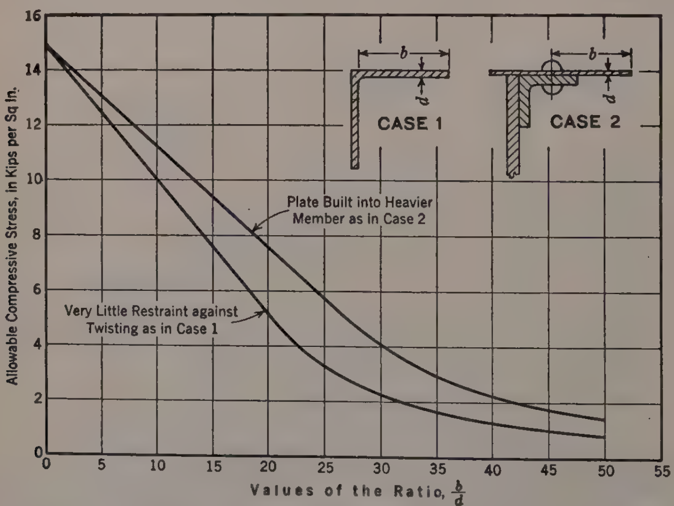


FIG. 41.—ALLOWABLE COMPRESSIVE STRESS ON OUTSTANDING PLATES AND LEGS, OF ANGLES OF ALUMINUM ALLOY

this basis. For projecting plates or legs of angles built into heavier members, as in the case of the flanges of girders or H-shaped columns having relatively thick webs, the coefficient, 4.0, may be reduced to 3.0 because such members offer a much higher degree of restraint against twisting as the outstanding legs or flanges begin to buckle. For intermediate degrees of restraint the coefficient should be selected between the limiting values, 4 and 3, so as to be consistent with the degree of restraint assumed.

When flat plates supported on either one side or on two sides are used as component parts of columns (see Fig. 41, Case 2) they should be investigated for stability in accordance with the foregoing recommendations, independently of the stability of the column as a whole. If it is found that the allowable stress on the plate is less than that for the column as a whole, the former, of course, will control the design of the column, and *vice versa.* The use of equivalent slenderness ratios makes this comparison very simple because it is only necessary to determine whether the equivalent slenderness ratio of the parts is less or greater than the effective slenderness ratio for the column as a whole. In many cases it will be found economical to make the equivalent slenderness ratio of the parts equal to the effective slenderness ratio of the member as a whole, although this cannot be stated as an infallible guide.

For bearing on rivets or on tightly fitted pins and bolts, a working stress of 26 kips per sq in. on the projected area is recommended. Tests have shown that this value represents a factor of safety of at least two against measurable permanent distortion of the hole, and more than three against utlimate failure, provided the edge distance from the center of the hole in the direction of stress is at least twice the hole diameter. For smaller edge distances the foregoing allowable stress (26 kips per sq in.) should be reduced proportionately.

For shear on aluminum (17S-T) rivets or on tightly fitted aluminum (17S-T) pins or bolts, a design stress of 9 kips per sq in. is recommended. This same limiting shear stress applies to the gross section of the webs of beams and girders, provided the shear on the net area on such webs does not exceed 12 kips per sq in., and provided thin webs are adequately stiffened. For shear on steel rivets an allowable stress of 13 kips per sq in. is recommended.

For the webs of beams and girders the maximum shear stress, in pounds per square inch, at the center of any stiffened panel should not exceed:

$$\frac{12\,000\,000}{\left(\frac{h}{d}\right)^2}\left[1 + \left(\frac{h}{l}\right)^2\right] \qquad (10)$$

in which d is the web thickness; h is the clear depth of web between flanges; and l is the clear distance between stiffeners, all in inches. When it is desired to investigate intermediate points in large panels subjected to a varying shear along the length, l may be defined as twice the clear distance to the nearest stiffener. When no stiffeners are used, l becomes very large and the quantity in the brackets approaches unity. Equation (10) not only provides a means of checking girder webs for buckling, but also permits the spacing of stiffeners to be determined to suit any condition of varying shear. It should not be used beyond the limiting value of 9 kips per sq in. Fig. 42 is a set of curves representing Equation (10).

Members which in ordinary service are subject to stress reversal, tension to compression, should be proportioned so that the larger stress plus one-half the smaller stress, does not exceed the basic allowable stress of 15 kips per sq in. This precaution in design will guard against fatigue failures under all ordinary service conditions, because it provides for a million cycles of reversal with an allowance for "stress raisers", such as rivet holes. Of course,

structures that are subject to a considerably larger number of reversals or that have sharp re-entrant corners, and other severe "stress raisers", should receive special treatment in design and cannot be classed with ordinary structures.

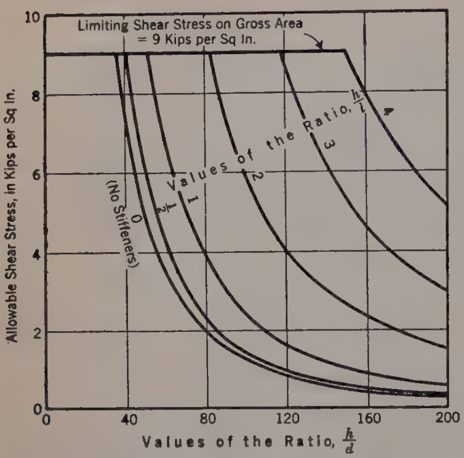


FIG. 42.—ALLOWABLE SHEAR STRESS ON ALUMINUM ALLOY (17S-T) GIRDER WEBS, EQUATION (10)

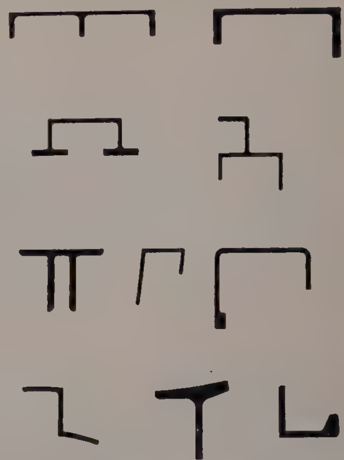


FIG. 43.—EXAMPLES OF ALUMINUM ALLOY SPECIAL SHAPES PRODUCED BY THE EXTRUSION PROCESS

The foregoing safe working stresses are intended to serve as a guide in the design of ordinary structures built of standard structural aluminum, in the same manner that the familiar design specifications for standard steel structures are used. The factors of safety are selected conservatively as indicated. In specialized fields of design these factors of safety may prove to be more conservative than necessary, in view of the exact knowledge of loads and other design conditions, and in such cases the engineer, of course, will use working stresses consistent with the nature of the problem at hand.

Impact allowances on aluminum alloy structures should be the same as those used for comparable steel structures. The entire question of dynamic loadings in ordinary structural design is still on a very unscientific basis, and the mere addition of an arbitrary percentage to the static live load stresses is only a crude attempt to provide a measure of safety against a little understood type of failure. Until a more scientific basis of designing for dynamic loadings comes into general use, there is no point in trying to make adjustments upward or downward in the conventional impact allowances to cover light weight, low modulus materials. Theoretically, of course, the relatively low modulus of elasticity makes aluminum alloys better suited to absorb energy within the elastic range, and this fact is readily demonstrated by tests of simple structural elements. A study of the results of tests and accidents involving more complex built-up structures, however, suggests that the behavior of such structures under dynamic loadings, either in the elastic range or beyond, may be more a function of design than of material, and that aluminum structures may be expected to withstand dynamic loadings at least as well as steel structures of comparable design.

In designing for light weight probably the most important single factor, is the ingenuity of the designer in selecting the best type of structure to carry the loads and meet the other controlling conditions. The designer and detailer must also be adept at arriving at the proper conclusion regarding such questions as the best distribution of metal in various members; when to substitute latticing for solid webs; when to use thin sections with many stiffeners as compared to thicker sections and fewer stiffeners; where to locate splices and connections; and when to use continuity to the best advantage. In all such matters the proper answers change from one structure to the next, depending upon the conditions of the design. The only trustworthy answer to any of these problems is obtained through a careful study of fairly complete comparative designs. Experience gained in the design of steel structures is not always a safe guide in determining how to arrive at the lightest weight in aluminum alloy structures.

The engineer designing in structural aluminum for the first time, is usually intrigued by the possibilities offered by the extrusion process for producing special shapes useful in structural design. In addition to the standard structural shapes, the user of structural aluminum has available an infinite variety of these special shapes, some of which are shown in Fig. 43, and many an aluminum structure owes at least part of its exceptional light weight and lower cost to the intelligent use of such shapes. On the other hand, it cannot be denied that the reverse may easily occur; that is, the novice in aluminum design may be over-enthusiastic about the possibility of extruded shapes, and burden a relatively small job with the cost of several extrusion dies where standard structural shapes might have served practically as well. It should not be difficult in any given case to determine the relative merits of special extrusions and standard shapes if the designer confers with the producer of the special shapes, so that if any extra costs are involved they can be evaluated properly early in the design.

It is a well-known fact that because of the lower modulus of elasticity, aluminum members deflect more than steel members of equal size under the same loading conditions. Occasionally, this fact has been a deterrent for engineers interested in the design of aluminum structures, and almost invariably the reason lies in the fact that the question of deflection is not faced squarely. There is a tendency on the part of some engineers to limit deflections entirely on the basis of their experience with existing steel structures rather than to make an effort to determine the allowable deflection, as an engineering problem. The writer fully appreciates that, in many instances, it is extremely difficult to determine just what the deflection requirements should be, but if minimum weight is to be attained in any structure it is essential for the designer to avoid all unnecessarily severe deflection restrictions.

In general, it is only live load deflections that need concern the designer because excessive dead load deflection can be corrected by proper camber. In addition, in light weight structures the dead load deflections are usually minimized by the weight savings. In members in which deflection becomes a

controlling feature of design it is generally advantageous, of course, to increase the depth, thereby distributing the metal to the best advantage to resist deflections. A simple illustration may help to show the advantages of increased depth; for example, a 5-in. aluminum I-beam for a given span and load will deflect approximately three times as far as the same size of steel I-beam weighing three times as much. If the 5-in. aluminum beam is replaced by a 7-in. aluminum beam, however, the deflection will be reduced to the same as that of the steel beam, whereas the weight is increased to a value which is still only one-half that of the 5-in. steel beam.

Sometimes in striving for the lightest and best possible structure, the engineer will discover that other structural metals, such as the alloy steels, can be used to advantage in some part of what would otherwise be an all-aluminum structure. This is particularly true in the case of highly stressed tension members, shear pins, parts subjected to severe abrasion, shallow beams in which the depth cannot be increased readily, and special high-stressed castings. No special precautions are required in such composite construction, except that the possibility of temperature stresses due to differences in coefficient of expansion, should not be over-looked. Although the magnitude of such temperature stress rarely approaches one-half the design stresses, even where the different metals are riveted together over a considerable length, an attempt should be made to avoid such stresses wherever possible by arranging the different metals in such a way that temperature stresses are minimized. It should also be remembered that when a composite structure is loaded, the stresses in adjacent parts which are riveted securely together will be proportional to the moduli of elasticity. For example, if an aluminum cover-plate is riveted to a steel beam, it will carry only one-third its full share of the load and will not strengthen the beam nearly as much as it would an aluminum beam under the same conditions.

In using various metals in combination in a structure, care should be taken to avoid the possibility of galvanic action which might accelerate corrosion at the points of contact of the different metals, either by breaking the contact with an insulating material, or by excluding all moisture through the use of thoroughly protective paint coatings. The latter precaution is the only one necessary in most cases, and from the excellent condition of existing aluminum structures in which steel rivets have been used, one may readily deduce that this protection is entirely adequate.

The writer would like to emphasize the importance of giving proper attention to non-strength members and secondary strength members in a light-weight structure. Too often when an engineer is studying the possibility of weight saving he is inclined to overlook the fact that a fair percentage of the total weight of his proposed structure lies outside the main strength members. Careful attention given to such items as bracing, walk-ways, walk-way supports, hand-rails, closing sheet, etc., will often be rewarded by surprisingly large additional weight savings, and any structure in which weight saving is important certainly deserves this extra attention. Sometimes it will be found that a study of these structurally unimportant parts will lead

to combining the functions of parts, thereby making possible the elimination of superfluous members and their connections. In any event, it is an important advantage of the light alloys that they permit weight savings in low stressed parts, which are very difficult to obtain in heavier metals that save weight only by virtue of their superior mechanical properties.

Weight Saving Possibilities

The question of when to use structural aluminum is an interesting one, which obviously cannot be treated adequately without going into a detailed and lengthy discussion. The writer will attempt only a brief review of the more important generalities. There are three questions to be answered: (1) What weight savings are possible? (2) What additional expense is incurred to obtain these weight savings? and (3) Can this extra expense be justified? Since structural aluminum weighs 35% as much as steel, a direct substitution, section for section, will lead to a weight saving of 65%, and this margin is often attained through exactly this procedure in actual practice. In most structures, however, the weight savings will range from 50% to 60%, because the designer finds it necessary, in certain members, to use slightly deeper or thicker sections than would be required for equivalent service in steel. The writer has made a study of the weight savings possible in various types of aluminum alloy structural members, tension, bending, and compression, assuming equal conditions of service, and finds that in the average structure in which dead load stresses are relatively small, theoretically, the weight of structural aluminum (17S-T) members should be about 45% that of the same members designed in structural steel for the same loads, a weight saving of 55 per cent.

In larger structures in which the dead load stresses become important in the design of the various members, the weight saved by the use of structural aluminum is greater due to an interesting pyramiding effect. When such a structure is made lighter, the decrease in weight causes a decrease in the dead load stresses which, in turn, permits an additional decrease in size of members, etc. Studies show that in these larger structures weight savings of 65% to 70%, and even greater, should be expected.

In discussing weight savings the use of percentages may lead to a certain degree of confusion. Throughout this paper the writer has used weight-saving percentages defined as follows:

$$\begin{matrix}\text{Percentage of}\\ \text{weight saved}\end{matrix} = 100\left(1 - \frac{\text{Total weight of aluminum members}}{\text{Total weight of equivalent steel members}}\right) \quad \ldots (11)$$

It does not apply to the over-all weight of structures, except in those instances in which the structural members themselves make up the entire weight of the structure; for example, consider the case of a dragline boom built entirely of structural aluminum and compared with a steel boom of exactly the same length and designed for the same service. The aluminum boom in this case might be expected to weigh 45% of the weight of the steel boom, provided both weights are for the structural parts only, exclusive of cables, sheaves, etc.

If the over-all weight of the dragline unit, including cables, machinery, and sheaves, is used in the comparison, the foregoing weight-saving percentage needs very radical revision. Other complicating factors in the interpretation of weight-saving percentages would be introduced into this same illustration if the aluminum boom was made longer than the steel boom to take advantage of the lower over-turning moment, or if only the outer two-thirds of the boom were constructed of aluminum instead of the entire boom. It is not the writer's purpose to enter into a discussion of these complicating factors, but it is necessary to emphasize that the weight-saving data introduced in this paper are intended to represent the differences in weight between any group of aluminum structural members and the same group of members designed in steel for equivalent service.

The price per pound of structural aluminum plates and shapes will average about fifteen times the corresponding price per pound of structural steel. Reduced to a volume basis instead of a pound basis the structural aluminum plates and shapes will cost about five times as much as those of structural steel. Therefore, comparing two structures having members of identical size throughout, one built of structural aluminum and the other of structural steel, the cost for material alone in the aluminum structure should be about five times that of the steel structure. For structures which in aluminum weigh 45% as much as the same structures designed in steel, the extra cost of the aluminum plates and shapes will be more nearly seven times that of the equivalent steel. If the extra cost is divided by the weight saved, the additional cost for material in the aluminum structures can be

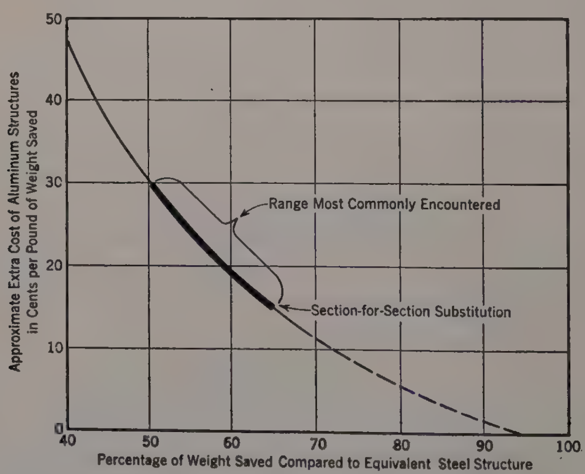


FIG. 44.—EXTRA COST OF STRUCTURAL ALUMINUM STRUCTURES COMPARED TO ORDINARY STEEL

expressed conveniently in terms of extra cost per pound of weight saved. The writer has made a study along these lines, and has plotted the results in Fig. 44 which shows how the extra cost per pound of weight saved decreases as the percentage of weight saved increases, the most common range being from

15 cents to 30 cents extra per lb of weight saved. Of course, there are many factors that may operate to invalidate the values indicated on this curve, but the writer believes that they will be found fairly representative for the ordinary structures encountered to-day.

Although the extra cost values given in Fig. 44 are for material only, they should also represent, quite closely, the over-all difference in cost of fabricated structures because experience has shown that the cost of fabricating any aluminum structure is practically the same as that for fabricating a comparable steel structure. Of course, structural aluminum, because of its light weight, may effect slight savings through lower handling costs, but these savings are frequently offset by the fact that workmen may tend to handle the aluminum more carefully to avoid spoilage.

Since aluminum structures are more expensive than equivalent steel structures, the question of when to use aluminum becomes largely one of justifying the extra cost. Obviously, many structures are built, in which there is no possibility of justifying an extra expenditure of even 10 cents for each pound of weight saved, and for the present these structures are outside the field of structural aluminum. There are many structures, however, in which 15 to 30 cents extra cost per lb of weight saved (and even more) is easily justified. Most of such cases, of course, fall in the classification of moving structures where operating economies are effected. The aircraft and transportation fields have been, and will probably continue to be, the most fertile field for the use of structural aluminum, but engineers are rapidly finding other fields in which the light metals are making a place for themselves in spite of the cost. Such fields include dragline and crane booms, traveling cranes, mine-hoist cages, ship superstructures, etc. One interesting field of use of the light structural metals is in applications where the reduction of dead weight permits the useful life of an existing structure to be extended to accommodate changes in loading conditions not contemplated in the structure as originally designed. Examples are found in the replacement of heavy bridge floors with new light-weight floors, and the use of light-weight traveling cranes in buildings not sturdy enough to support a conventional crane of the required capacity.

As a specific example of the economics of the use of structural aluminum, the writer would like to cite the case of dragline booms, such as are used in levee construction. In this field, the weight saved by the use of aluminum is used most effectively to increase the length of the boom, and, hence, the operating range of the dragline unit. It has been found that a 150-ft steel boom, weighing 33 000 lb, can be replaced with a 175-ft composite aluminum-steel boom weighing about 23 000 lb, the weight saving being accomplished by using 12 000 lb of structural aluminum in the outermost 140 ft of the length. The resulting light-weight boom will handle the same size of bucket as the steel boom, with no greater overturning moment on the machine, and with no decrease in the swing speed. Because of the extra 25 ft in length there is less rehandling of material for the same capacity of bucket so that the speed of the work is increased about 10%, resulting in a saving of about 1 cent per cu yd of earth moved, and, thereby, increasing the earnings by about $1 000

per month per machine. The extra cost of the light-weight boom is thus defrayed in about five months, insuring an adequate return on the additional investment.

No discussion of the costs of aluminum alloy structures is complete without some mention of the scrap price of the metal, which in the case of fabricated structural shapes and plates is about 12 cents per lb at the present time. This high return value of the metal does not affect the extra first cost of structures, of course, and hence has no affect on the values shown in Fig. 44. It should not be overlooked in the general study of the economics of light-weight design, however, particularly on structures having relatively short periods of usefulness due to rapid obsolescence.

Conclusion

Structural aluminum is firmly established in those fields in which it is now being used successfully. Its future in other fields is a matter which lies almost entirely in the hands of designing engineers interested in producing structures which best serve their intended purposes. The writer has tried to indicate that the choice of structural aluminum is almost entirely an economic problem rather than a structural one; that is, structural aluminum may be utilized with confidence in any case where its cost can be justified.

There are many factors, of course, which may affect the future economic status of structural aluminum. Such factors include metallurgical or manufacturing changes which may effect the cost of the metal, the development of new alloys having better mechanical properties, or are otherwise better adapted for structural purposes, new developments in welding, or other fabrication methods, etc. Although some progress is being made at present along many such lines, the writer sees no immediate prospect for any radical change in the status of structural aluminum, and believes that its progress in the structural field in the near future will be simply a continuation of the steady, healthy growth which has marked the development to its present stage of usefulness.

Acknowledgments

The writer is indebted to R. L. Templin, M. Am. Soc. C. E., and to R. G. Sturm and B. J. Fletcher, Assoc. Members, Am. Soc. C. E., for assistance and suggestions in the preparation of this paper.

MAGNESIUM ALLOYS AND THEIR STRUCTURAL APPLICATION

By A. W. Winston,[98] Esq.

Synopsis

The combination of light weight with the improved mechanical properties of the newer magnesium alloys makes these materials of increasing interest to the civil engineer. Since 1933 or 1934, production costs have been reduced, and properties have improved to the point where the alloys are finding many successful applications formerly considered impractical for the material.

In this period a number of important applications have developed in the transportation field. As might be expected, because of the necessity for light weight, the airplane industry was the first to adopt these alloys in all types of service. Sand castings of magnesium alloys are used universally in airplane motor and starter castings, in landing wheels and brakes, and as brackets and fittings in the airplane structure. A number of truck and trailer bodies have been built of sheet and structural shapes, resulting in reduced weights and increased operating economy for their owners.

Although they are not used extensively as yet in structural engineering, these alloys give promise of development in this field of application. The purpose of this paper, therefore, is to present the characteristics of the standard magnesium alloys in which the civil engineer is most likely to be interested, in order that he may arrive at a proper appreciation of the possibilities in their use.

Introduction

In changing from one material to another it is very necessary that careful consideration be given to all the factors involved. The success of a design may depend upon the attention given to details which are automatically provided for by the engineer when working in steel, but which may escape attention when he is starting to use a new material.

The metallurgical aspects of magnesium alloys are presented in Part II of this Symposium. Not less important than a knowledge of these fundamental properties and characteristics is a practical knowledge of the performance of actual beams and columns, of correct shop and field methods of fabricating, and of the protective measures to be applied to insure long and satisfactory service.

[98] The Dow Chemical Co., Midland, Mich.

The designer contemplating the use of magnesium has available a wide variety of alloys and forms which are adaptable to many specific uses. The manufacture of castings, structural shapes, sheet, and forgings, and the properties obtainable in these forms, have been described in detail in the paper, by Messrs. Zay Jeffries, C. F. Nagel, Jr., and R. T. Wood, entitled "Light-Weight Structural Alloys."[99] On comparing the properties of the standard, cast magnesium alloy with those for cast iron and the cast aluminum alloys (Table 25), it will be observed that the former compare exceptionally well on the basis of unit strength. Although not quite equivalent on a volume basis, the wrought alloys possess very favorable strength-weight ratios. Usually, it is possible to increase the sections slightly to secure the desired stiffness and strength without sacrificing the weight advantage of the magnesium alloys.

TABLE 25.—COMPARATIVE PROPERTIES OF MAGNESIUM AND OTHER STRUCTURAL ALLOYS

Item No.	Alloy and temper (1)	Specific gravity (2)	Weight, in pounds per cubic foot (3)	Tensile strength, in kips per square inch (4)	Yield strength, in kips per square inch (5)	Percentage elongation in 2 in. (6)	Modulus of elasticity, in kips per square inch (7)	Endurance limit, in kips per square inch (8)	Coefficient of thermal expansion, in inches per inch per degree Fahrenheit (9)	Specific Stresses,† in Kips per Square Inch: Tensile strength (10)	Yield strength (11)	Endurance limit (12)
	(a) Cast Metals											
37*	Magnesium:¶											
	Heat-treated	1.83	113	36	13	11	6 500	10.0	0.000016	19.7	7.1	5.5
	Heat-treated and aged	1.83	113	40	20	5	6 500	10.0	0.000016	21.8	10.9	5.5
39	Aluminum copper‡	2.83	178	22	14	2	10 000	7.5	0.000013	7.8	5.0	2.7
40	Aluminum Copper:											
	Heat-treated§	2.77	174	36	22	4	10 000	6.0	0.000013	13.0	7.9	2.2
41	Gray cast iron‖	7.2	450	40	..	..	18 000	20.0	0.0000060	5.6	....	2.8
42	Cast steel; 0.30% annealed	7.86	490	76	42	..	29 000	33.0	0.0000066	9.7	5.3	4.2
	(b) Wrought Metals											
31*	Structural magnesium**	1.80	112	43	30	17	6 500	17.0	0.000016	23.9	16.7	9.5
43	Structural magnesium††	1.80	112	45	35	12	6 500	....	0.000016	25.0	19.4	...
44	Duralumin	2.79	174	60	36	20	10 000	15.0	0.000012	21.5	12.9	5.4
45	Low-alloy steel	7.85	490	80	60	22	29 000	40.0	0.0000066	10.2	6.8	5.1
46	Chrom-molybdenum steel	7.85	490	125	90	13	29 000	70.0	....	15.9	11.5	8.9

* See corresponding Item Numbers in Table 10 (see p. 1232). † Columns (4), (5), and (8), respectively, divided by Column (2). ‡ Alloy No. 12, Specification 30, Handbook, Soc. of Automotive Engrs., 1936 Edition. § Specification 38, Soc. of Automotive Engrs. ‖ Specification A 48–35 T,, Class 40, Am. Soc. for Testing Materials. ¶ Known commercially as AM 265. ** Known commercially as AM 57S. †† Alloy X.

DESIGN FACTORS FOR MAGNESIUM ALLOY STRUCTURES

In considering the more important properties of magnesium alloys and their effect upon structural design, reference should be made to Table 25 and Table 26 for details of the comparative properties discussed. The relative bending properties listed in Table 26 are based on the physical properties given in Table 27. The standard formulas for rectangular beams were used in making the calculations.

[99] See p. 1211.

TABLE 26.—Relative Strength and Stiffness in Bending of Sheet Metals (Widths Constant)

Comparison	Material	Using Steel on a Basis of 100 for Comparison				Using Aluminum Alloy on a Basis of 100 for Comparison			
		Thickness 100	Strength 100	Stiffness 100	Weight 100	Thickness 100	Strength 100	Stiffness 100	Weight 100
Equal thickness	Aluminum alloy.	100	58	34	36	.....	......	.....	......
	Magnesium alloy	100	50	22	23	100	86	65	64
Equal strength.	Aluminum alloy.	131	100	77	47	.....	......	.....	......
	Magnesium alloy	141	100	62	32	108	100	82	69
Equal stiffness.	Aluminum alloy.	143	118	100	51	.....	......	.....	......
	Magnesium alloy	165	136	100	38	115	114	100	74
Equal weight. .	Aluminum alloy.	281	458	755	100	.....	......	.....	......
	Magnesium alloy	436	950	1 823	100	155	206	242	100

Specific Gravity.—The specific gravity of the present commercial magnesium alloys will average nearly 1.8, which may be compared to 2.8 for aluminum alloys and 7.85 for steel. The weights per cubic foot are, respectively, 112, 175, and 490 lb. For equal volumes the saving in weight by the use of magnesium alloy in place of aluminum would be 63 lb per cu ft, or 36%,

TABLE 27.—Physical Properties of the Alloys Compared in Table 26.

Material	Specific gravity	Tensile yield strength, in kips per square inch	Young's modulus of elasticity, in kips per square inch
Steel....................................	7.85	60	29 000
Aluminum alloy..........................	2.79	35	10 000
Magnesium alloy.........................	1.80	30	6 500

and in the case of steel, the saving would be 378 lb per cu ft, or about 77 per cent. In general, not all this theoretical saving can be realized when allowance has been made for differences in strength and modulus of elasticity. With castings, however, these savings are attained frequently. An example of this is found in some large ventilating fans, 12 ft in diameter. When constructed of aluminum alloy, the castings weighed approximately 900 lb. The use of magnesium alloy castings made from the same patterns has lowered the casting weight to less than 600 lb, a net saving of more than 300 lb. The lighter fan has permitted the use of a smaller motor because of the lessened starting load and has reduced, considerably, the cost of the unit. Patterns used for aluminum or cast iron ordinarily may be used for magnesium alloys without modification other than adjustment for shrinkage and increase in the radii of fillets.

Thermal Expansion.—The coefficient of thermal expansion of magnesium alloys is generally accepted to be 0.000016 in. per in. per degree Fahrenheit. This value is slightly greater than that for aluminum alloys (0.000013) and considerably greater than that for steel (0.0000066), or that for cast iron

(0.000006). This difference becomes important when structures subject to temperature changes are being designed, in which magnesium alloy members may be rigidly connected to members of iron or steel; for example, a magnesium alloy floor-plate 10 ft long may be riveted to a steel framework made necessary by space limitations. If such a structure is subjected to a temperature rise of 100° F, the difference in expansion of the two materials will be almost $\frac{1}{8}$ in., an amount sufficient to cause partial shear of the rivets or serious and possibly permanent deformation of the structure. It is important, therefore, that the designer recognize this possibility and provide for it by the elimination of contributing causes. The use of shorter members, expansion joints, and care in the method of attachment will do much to make such composite structures practicable.

Modulus of Elasticity.—The generally accepted value for this constant is 6 500 000 lb per sq in. As this is somewhat lower than the modulus for aluminum (10 000 000), and considerably lower than that for steel (29 000 000), it is necessary to take this fact into consideration in re-designing structures for magnesium alloys.

To secure equal stiffness in bending, which usually is desired, it is necessary to increase the moment of inertia of the section. This may be accomplished with a relatively small increase in dimensions since the moment of inertia varies as the cube of the depth. By designing to equal stiffness, the yield strength will be increased over that of the steel or aluminum member being replaced, retaining very significant weight savings, as indicated in Table 26, and described in greater detail under the heading, "Beams."

The differences in moduli of elasticity must also be considered in designing composite structures to insure correct load-transfer conditions. Thus, if a magnesium alloy tension member is riveted to a steel section, both pieces should be adjusted in size to secure equal elongation between rivets when under operating load. Otherwise, the unequal elongation will throw an excessive load upon certain rivets, possibly resulting in their failure or in the elongation of the rivet holes.

Mechanical Properties.—An appreciation of the possibilities in the use of magnesium alloys is obtained by reference to Table 25 in which the comparative properties of some of the important structural metals are given. As previously noted, cast magnesium alloys have mechanical properties equal to those of the cast aluminum alloys or of cast iron on an equal volume basis. When compared on an equal weight basis, both the cast and wrought magnesium alloys are equal or superior to the other metals in the cast or wrought conditions.

Proportional Limit and Yield Strength.—When magnesium alloys are subjected to the usual tensile tests, there is a gradual breaking away from the modulus line, making determination of tangential proportional limits somewhat difficult. A suggested method is to consider the proportional limit as the stress where the stress-strain curve deviates 0.01% from the modulus line.

Similar to aluminum alloys but unlike steel, magnesium alloys do not exhibit sharp yield points. The yield strength is now accepted as the stress

where the stress-strain curve deviates 0.2% from the modulus line. The usual value obtained for the proportional limit is from 50 to 60% of the yield strength.

A peculiar characteristic of wrought magnesium alloys, recognized by investigators for a number of years, is the low compressive yield strength compared to the tensile yield strength. This value, determined at 0.2% deviation from the modulus line as in the case of the tensile yield strength, is about 75% of the latter value. This ratio is subject to some variation and will be affected by mechanical working, composition, and heat treatment.

TABLE 28.—PROPERTIES OF EXTRUDED SECTIONS OF STRUCTURAL MAGNESIUM ALLOYS

Item No.*	Tensile strength, in kips per square inch (1)	Tensile yield strength, in kips per square inch (2)	Percentage elongation in 2 in. (3)	Compression yield strength, in kips per square inch (4)	Flexure yield strength, in kips per square inch (5)	Brinell hardness number (6)	Tensile strength, in kips per square inch (1)	Tensile yield strength, in kips per square inch (2)	Percentage elongation in 2 in. (3)	Compression yield strength, in kips per square inch (4)	Flexure yield strength, in kips per square inch (5)	Brinell hardness number (6)
	(*a*) 3 by 4-Inch by $\frac{3}{16}$-Inch Angles						(*c*) 2-Inch Channels					
31	43.4	24.5	19.7	16.3	.....	58	41.5	21.5	19.5	15.3	.....	57
35	44.2	25.6	13.1	18.9	.....	67	46.0	29.4	13.1	18.4	.....	56
43	43.1	28.6	17.0	23.7	.....	69	44.5	30.0	15.0	27.4	29.5	69
	(*b*) 3-Inch I-Beams						(*d*) 4-Inch Channels					
31	43.3	27.1	13.5	18.3	.....	....	42.0	24.2	18.0	18.5	.....	58
35	43.4	29.0	8.0	.....	.....	54	42.7	26.5	12.2	16.4	.....	55
43	45.2	29.6	12.5	22.0	24.1	64	44.3	31.0	9.8	24.3	23.0	72

* Item numbers corresponding to those in Table 10 (see p. 1232).

In Table 28 are given the properties of the alloys in which wrought shapes are available, showing the progressive development, since 1934, in the direction of securing improved yield strength values. It has been noted that the methods presenting the best prospects for improvement along this line are those involving the heat treatment of wrought sections. The properties given in Table 28 are those obtained on structural sections. The yield strengths of small diameter bar stock as given in Table 25 are slightly higher than those of structural shapes, apparently due to more uniform conditions of flow through the extrusion die.

Fatigue Endurance Limit.—A noteworthy feature of magnesium alloys, particularly in the wrought condition, is the high fatigue endurance limit. Thus, it is not unusual to have an extruded magnesium alloy (AM57S),[100] of the same composition as that given in Item No. 31, Table 25, exhibit a fatigue endurance limit of 19 kips per sq in. with a proportional limit slightly less than this value, and a yield strength of about 33 kips per sq in. Endurance limit data probably will find increasing use in design as experience is obtained with actual service applications.

[100] The symbols in parentheses denote the trade designation by which the alloy is commonly recognized.

Bearing Strength of Sheet.—The bearing strength of magnesium alloy sheets is rather high compared to the tensile strength of the material, and in the case of a hard-rolled magnesium alloy (AM53S)[100], will run from 60 to 70 kips per sq in., whereas the annealed material will be from 55 to 65 kips per sq in. One-fourth of these ultimate values are considered to be safe design values.

Beams.—Magnesium alloy beams deflect appreciably with increasing load above the yield strength until failure finally occurs as stresses approach the ultimate tensile strength. This statement applies to simple beams, such as rectangular sections which fail by yielding of the material. The yield strength of a magnesium alloy beam is considered to be the point at which the outermost fibers are elongated 0.2% beyond the modulus line, and may be determined either by the use of strain-gages or by computing the deflection corresponding to this 0.2% permanent deformation of the outer fibers. This concept applies to the beams the same interpretation of yield strength as that used in the ordinary tensile or compression test on the material. Due to the reinforcing action of the material near the neutral axis, the flexural yield strength as determined in this manner will exceed the tensile or compressive yield strength of the material.

The most important factor that will interfere with the foregoing conclusion is that due to the form of the section. As the sections are made more slender, failure may occur through twisting, buckling, or flange crippling. In such cases, the yield strength of the beam may fall below the yield strength of the material in the outer fibers. Because of this fact, it is suggested that the design values be limited to one-half the flexural yield strengths as determined by actual tests of the sections under consideration. Examples of such tests are given in Table 28 for several typical sections.

Very few data have been obtained on the lateral buckling of compression flanges of beams in the relation of unsupported length to width of flange. Little information is available on the buckling of beam webs and on the crippling of compression flanges. Although most of the I-beam, channel, and plate girder tests conducted to date have failed by lateral buckling, the maximum stresses were about the same as those developed by similar sections which failed by compression yielding of the flanges.

Under the heading, "Modulus of Elasticity," attention was directed to the stiffness characteristics of magnesium alloy sections compared to those of aluminum alloy or steel sections. The relative effects of specific gravity, yield strength, and modulus of elasticity for these three materials on the design of rectangular beams have been computed and are given in Table 26. By reference to this table it will be observed that, on the basis of equal stiffness, the comparison is very favorable to magnesium alloys; for example, for equal stiffness, the magnesium alloy beam will be 65% thicker and 36% stronger than the corresponding steel beam, with a weight saving of 62 per cent. Compared to an aluminum alloy beam of equal stiffness, a magnesium alloy beam will be 15% thicker, 26% lighter, and 14% stronger.

The situation is slightly more complex when one wishes to substitute magnesium alloy structural sections for those of aluminum or steel. If the mem-

ber to be replaced is a steel beam, it is generally desirable to choose a geometrically similar magnesium alloy section having a moment of inertia about four and one-half times as great as that of the steel beam. This rough rule will provide a member of the same stiffness, of considerably greater strength, and of less than one-half the weight.

In like manner, an aluminum section may be replaced by a geometrically similar magnesium section having a moment of inertia about one and two-thirds as great as that of the aluminum section, resulting in a member about 20% lighter, but having the same strength and stiffness.

If wood is to be replaced by magnesium alloy with equal stiffness, the metal section should have a moment of inertia at least one-fifth that of the wood.

The substitution of magnesium alloy beams for steel or aluminum members in composite structures should be made on the basis of equal stiffness, as better load transfer conditions can be maintained with an increase in the strength of the assembly.

Columns.—Magnesium alloy columns act much the same under loads and are as susceptible to design calculations as columns of other materials. The strength of such columns is dependent upon the following factors: (1) Compressive yield strength; (2) ultimate compressive strength; (3) modulus of elasticity; (4) size; (5) shape; (6) end conditions; (7) loading conditions; and (8) length. Perhaps the most important of these factors are the compressive yield strength and the modulus of elasticity.

The compressive yield strength is the governing factor for most short columns used in structural work. As discussed under the heading, "Proportional Limit and Yield Strength," the compressive yield strength of standard magnesium alloy sections has undergone considerable improvement since 1933. Recent determinations for this value during a series of column tests on a variety of sections of magnesium alloy (see Item No. 43, Table 25 and Table 28), have indicated a value of 24 kips per sq in. for the compressive yield strength. It was found that the results of these column tests were closely approximated by a modified Rankine-Ritter formula:

$$\frac{P}{A} = \frac{s_y}{1 + Q\left(\frac{L}{k}\right)^2} + \frac{80\,000}{\frac{L}{k}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

in which P = a load, in pounds; A = cross-sectional area, in square inches; s_y = compressive yield strength = 24 000 lb per sq in.; L = length of column, in inches; k = least radius of gyration; and Q = Ritter's constant:

$$Q = \frac{s_y}{C\,\pi^2\,E} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

in which C = a fixation coefficient and E = the modulus of elasticity = 6 500 000 lb per sq in. The results of the tests are plotted in Fig. 45. Although service experience is somewhat lacking in the application of these data, it is suggested that design values should not exceed one-half the unit strength given.

Because of the limited space available in this paper, it will be possible to give only a single example of the column characteristics of magnesium alloys compared to those of aluminum alloys or steel.

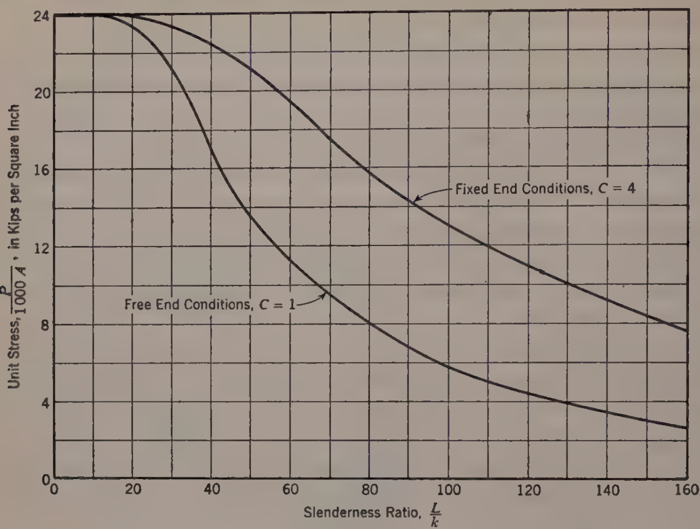


FIG. 45

Assume that a load of 20 000 lb is to be supported by a single fixed-end solid column of round cross-section and a length of 60 in. On computing the necessary diameters, using the published design values for steel and aluminum alloy and one-half the yield strength values for magnesium alloys, as given in Fig. 45, the results given in Table 29 were obtained.

TABLE 29.—COMPUTATION OF COLUMN DIAMETERS

Material	Diameter, in inches	Weight of column, in pounds	Percentage of steel weight saved
Steel	1.72	39.4	..
Aluminum alloy	1.89	17.0	57
Magnesium alloy*	2.10	13.7	65

* Alloy X; see Item No. 43, Table 25.

Some variation from these weight ratios will be experienced when other specific cases are estimated, due to changes in the relative permissible stresses, which will be dependent upon the shape of section, $\frac{L}{k}$-ratios, end conditions, eccentricity of loading, etc. In general, however, the order of weight difference will be similar to that given in Table 29.

FABRICATION METHODS

The successful application of magnesium alloys has been dependent largely upon the development of suitable fabrication methods. Among the important

features of good shop practice are the results and conclusions of many years of experience in the production and use of a wide variety of articles made of magnesium alloys, including castings and built-up assemblies using structural shapes and sheets. As with any other structural material, careful attention to small details has been an important factor in the success of these applications.

Machining.—The machinability of the magnesium alloys is unexcelled by that of any other structural metal. A fine smooth finish is readily secured with no tendency to drag, tear, or chip out. Experience in the average machine shop has proved that practically all machine tools can be run at their maximum speeds with feeds to the full capacity of the machine. High-speed steel or tungsten carbide tools are generally recommended because of the long life between grindings.

Ordinarily, no cutting compound or lubricant is required as heavy cuts and feeds may be taken at high speed without excessive heating of cutting tools or work. A coolant is sometimes advisable on high-speed screw machine operations where a very fine finish is desired.

The power requirement in machining is less than that for any other metal. Comparative tests between magnesium and other alloys show that the power required will be about one-half that for aluminum alloys and brass, one-third that for cast iron, and approximately one-fifth that for steel.

Because of this excellent machinability, manufacturing costs frequently are reduced in spite of a larger cost for the rough casting or forging, particularly where the machining expense is a large proportion of the total cost. Machine production also is increased and, at the same time, operating and maintenance costs are lowered.

Forming.—Magnesium alloy sheet and shapes can be given a moderate amount of cold working, but when sharp bends are required it is necessary to heat the work and the tools. A temperature of 500 to 700° F is best for hot forming. The work may be heated locally with a torch, but a better method is to heat the part in an oven where the temperature can be controlled.

Because of the cold-hardening characteristic of the metal, spinnings, stampings, and drawings are limited to those of liberal bend radii in which only moderate deformation is required. In this case, also, heating of the stock and tools will permit more difficult operations. Tools should be clean, smooth, and well lubricated. Lard oil is generally satisfactory for the purpose.

Although the application of some of these processes may involve slight changes from present practice on other materials, most of these problems are capable of solution with a little expenditure of study and effort.

Riveting.—Rivets made of aluminum alloys, with properties given as Items Nos. 1, 2, and 6, in Table 6[101] (Alloys 2S, 3S, and 17S), may be used in magnesium alloy structures. The first two compositions are soft, are easily headed cold, and can be used where strength is not important. Item No. 6 of Table 6[101] (17S or AM55S), should be used where higher stresses are encountered. Alloy 17S has the higher strength, but must be heat treated just prior

[101] See p. 1217.

to driving. Alloy AM55S is especially recommended for use in magnesium alloy structures as the rivets may be driven cold and have the advantage of minimizing the possibility of galvanic corrosion under certain atmospheric conditions. The design of riveted joints, as with other metals, is based upon the shearing strength of the rivet and on the tensile and bearing strengths of the sheet. The use of steel rivets occasionally may be desirable in locations not subject to weathering where strength requirements or convenience in driving may make them more practical than aluminum alloy rivets. To prevent galvanic action, the steel rivets should be dipped in an insulating primer or compound before driving. One-fourth the bearing strength, or about 15 kips per sq in., may be taken as the safe design value for magnesium alloy (Item No. 26, Table 10).[102]

Welding.—Considerable success has been attained during the past few years in the development of welding processes for magnesium alloys. The methods generally in use at this time are gas welding and electric resistance welding. In oxy-acetylene welding a special flux must be used, while the filler rod should be of approximately the same composition as the part being welded. The presence of traces of flux left in the weld will tend to promote subsequent corrosion, and it is necessary, therefore, that considerable care be exercised during the welding operation to prevent such inclusions. After welding, the parts should be thoroughly cleaned in hot water and treated by immersion in the chrome-pickle solution (described under the heading, "Surface Treatment and Painting"), and then painted. Because of the difficulty in treating large structures, the acetylene welding process should be limited to articles and assemblies that can be cleaned adequately.

Electric spot welding has been used in a number of cases. It offers the advantage of speed and economy, and is applicable to the joining of sheet metal and extruded shapes. The following data give the shear strengths from tension tests of single spot welds joining strips of Magnesium Alloy AM53S (Item No. 26, Table 10)[102]:

Single sheet thickness, in inches	Shear strength, in pounds per spot
$\frac{1}{32}$	500 to 600
$\frac{1}{16}$	900 to 1 000
$\frac{1}{8}$	1 600 to 1 900
$\frac{1}{4}$	1 900 to 2 200

Surface Treatment and Painting.—Under ordinary conditions of atmospheric exposure, magnesium alloys have proved remarkably stable over periods of years. The surface, particularly if polished or buffed, gradually tarnishes and becomes covered with a thin gray oxide film. Some powdering and roughening of the surface occurs in heavy industrial areas or in locations of continuously high humidity. This corrosion is a very slow process, however, being measured in terms of years, and is very much slower than the corresponding rusting of mild steel in the same atmosphere. In saline atmospheres along the sea coast, corrosion may become more serious and necessitate preventive measures.

[102] See p. 1232.

Because of the difficulty in controlling the location and conditions under which an article will be used, it is always recommended that magnesium alloy assemblies or parts be given suitable paint protection. It has been demonstrated that, with proper care, paint systems on magnesium alloys will have a useful life in service as long and as satisfactory as on any other structural metal.

The surface treatment prior to painting is of great importance in the development of a protective paint system. The application of paint coats to the bare metal will result in unsatisfactory adhesion. A chemical treatment known as the chrome-pickle has been developed which, when applied to the metal, imparts to it definite corrosion inhibitive characteristics and a surface etch which promotes satisfactory mechanical bond. The chrome-pickle treatment consists of a dip for 30 sec at room temperature in a bath containing nitric acid and sodium bichromate. The yellowish irridescent coating formed by this treatment possesses good bond to the metal and in conjunction with suitable primers insures lasting adhesion of the paint system.

As a result of thousands of exposure tests, a number of paint schedules have been developed for a variety of service conditions, combining adequate protection with attractive decorative characteristics. A primer is extremely important and must be carefully selected for the particular service to be encountered. For general outdoor exposure, a primer recently developed by the United States Navy (Navy Specification P-27) has been very satisfactory, as it combines excellent adhesion with the desired inhibitive characteristic due to the zinc chromate pigment.

The last few years have seen the development of a number of very superior paint finishes compared to the old oil paints. They possess remarkably good weather resistance and imperviousness to moisture, due largely to the use of synthetic resins in their manufacture. Because of these qualities, they have proved to be very satisfactory for finishes over magnesium alloys. For ordinary atmospheric exposure, two coats of the pigmented enamels are generally satisfactory. For severe exposure conditions, it is recommended that three finish coats be applied, consisting of a varnish developed by the United States Navy (Navy Specification V-10*c*), containing $1\frac{1}{2}$ lb per gal of aluminum powder.

In building assemblies from magnesium alloy structural shapes and sheets which will be exposed to the weather, care should be taken to avoid pockets that could entrap water. Enclosed areas should be provided with drainage and good ventilation, and should be given at least one coat of approved primer. Faying surfaces should be primed and allowed to dry before assembly. In locations where magnesium alloy surfaces will be in contact with wood or dissimilar metals, additional treatment with bituminous paint is recommended. A heavy sealing compound should be used if there is a possibility of water entering the joint.

Numerous structural magnesium parts and assemblies, such as motor castings, airplane wheels, and truck and trailer bodies, have proved that properly applied paint systems will give adequate protection. The use of

magnesium alloys in structural applications now appears practicable (from the protection viewpoint) even in the more severe exposures along the sea coast.

Conclusion

The development of structural applications for magnesium alloys is dependent upon the economy possible through their use. It is a question of balancing initial, fabrication, and maintenance costs against the benefits of saving in weight. Castings of structural magnesium alloys already have demonstrated their ability to compete successfully with older metals and to-day are rendering satisfactory service in hundreds of applications. These castings are of excellent quality and in most cases the cost is very little, if any, more than that of the heavier aluminum castings of the same quality.

With the introduction of stronger alloys and improved fabrication processes, the way is now open for the development of additional structural applications. A beginning has been made in the transportation industry, and it is logical to expect interest to continue in this field. On account of the relatively high cost compared to steel, and the lack of need for weight saving, it is improbable that the near future will see much use of magnesium alloys in stationary structures. On the other hand, the civil engineer is most likely to use them in reducing the weight of his equipment, in his crane booms, scaffolds, ladders, and other portable tools, to which magnesium alloys can contribute increased load capacity, strength, and convenience.